Second Pacific Area Meeting Papers

DESIGN AND TESTS OF BUILDING STRUCTURES

Symposiums on
Seismic and Shock Loading
Glued Laminated and Other Constructions

Published by the
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Seismic and Shock Loading
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Presented at the
SECOND PACIFIC AREA NATIONAL MEETING
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FOREWORD

This publication consists of two symposiums, both of which are of equal value and interest to those in the building construction field.

The continued recurrence of earthquakes in the West Coast Area have shown they need not result in the destruction of a building if proper design is used. The papers on this subject present valuable information on tests and design.

Glued-laminated structural members are being increasingly utilized for building construction with the advancement in quality of wood adhesives. The papers in this field discuss factors affecting strength and design principles.

The papers and discussions of these two symposiums were presented in two sessions of the Second Pacific Area National Meeting of the American Society for Testing Materials held in Los Angeles, Calif., September 16–21, 1956.

Both symposiums were sponsored jointly by Committees D-7 on Wood and E-6 on Methods of Testing Building Constructions. Mr. Ernest Maag, California State Department of Public Works, presided over the Session on Building Design for Seismic and Shock Loading. Mr. F. J. Hanrahan, American Institute of Timber Construction, served as chairman of the Session on Glued-Laminated and Other Wood Constructions.

NOTE.—The Society is not responsible, as a body, for the statements and opinions advanced in this publication.

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SYMPOSIUM ON BUILDING DESIGN FOR SEISMIC AND SHOCK LOADING

INTRODUCTION

Mr. L. J. Markwardt:[1]

The design of buildings and structures traditionally necessitates consideration of all types of loading encountered—live loads, dead loads, wind loads, vibration, earthquake shock, and blast forces. The relative importance of the different forces varies from region to region and state to state, so that it has remained for cities and states to establish code requirements essential for regional and local conditions.

Earthquakes are encountered throughout the world, but it remained for the Long Beach shock in 1933 to dramatize the potential hazard of many types of construction and particularly school buildings. As a result of public demand, legislation was passed in California establishing design requirements to meet local conditions for school and other buildings, and procedures for design approval. As a result, the role of the structural engineer has assumed increasing importance, and California has the largest and most active Structural Engineers Association in the country.

Wood has assumed increased importance in earthquake resistant construction, but some of the problems encountered showed the need of additional design data. As a result an extensive research program was developed covering the evaluation of the strength, stiffness, and rigidity of large diaphragms representing full-scale sections of buildings. Different types of bracing and joint details were studied. The results of these extensive studies have just recently become available. The Second Pacific Area National Meeting at Los Angeles in 1956 offered an appropriate opportunity to present a review of this subject matter. The "Symposium on Seismic and Shock Loading" was accordingly developed under the joint sponsorship of ASTM Committees D-7 on Wood and E-6 on Methods of Testing Building Constructions in carrying out the objective of the Society in furthering the promotion of knowledge of materials of engineering.

Chairman Ben Benioff:[2]

The interest in seismic or earthquake effects is not only a part of the structural engineer's problems relating to buildings, but, it is applicable to all structures—bridges, towers, aqueducts, or pipelines, to name some of the other elements effected by lateral force.

Other states have provisions in their codes for providing resistance to winds. California has provided for the lateral force, which ever is the greater—seismic or wind.

Of more concern to the structural engineer recently has come the problem of shock loading, or, in simpler terms, bomb blast. One thing that is quite sure in this field, in the realm of nuclear blast, is that no one is going to worry about most superstructures at the center or near the center of the target. But, at varied distances from the orbit, there are areas in which the intensity of the blast has the same effect as any other major lateral force. It is here that savings of our structures will be effected by means of adequate design. It has been stated by several well-known authorities in this field, Mr. George Housner, for one, that the design procedure used today by California engineers for design against seismic loading is probably as good an approach to the subject as any we have today. Factors will, of course, vary with the degree of resistance to be adopted. Earthquakes can, and have, occurred throughout the world. It is also known that shocks of varying intensity have occurred throughout the United States. Some have been very severe. But, it is also known that of all the heavy populated areas in this country, California is more subject to seismic disturbance.

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[1] Assistant Director, U. S. Forest Products Laboratory, Madison, Wis.; Chairman, Committee D-7.

[2] Consulting Structural Engineers, Calif.

It is recognized that a condition must be faced from which there is no escape. Earthquakes have occurred as far back as records have been kept; they will continue through our lifetime and far into the future. They are dangerous solely because buildings and houses are erected which can be shattered or shaken down. In themselves, earthquakes have little hazard to human beings. They are dangerous only when structures are poorly built so that they can be damaged or wrecked. It is just as easy to build them so they will not be shaken down. There is ample proof in the past that it can be done and is economically feasible. An earthquake can be made an interesting but not a dangerous occurrence.

As a matter of interest, some of the shocks of the past are recalled to mind. The three outstanding great shocks are:

1857, January 8–9, at Fort Tejon, northwest corner of Los Angeles County, felt from Sacramento into Mexico—probably one of the most violent earthquakes known to have occurred in California. There was visible movement on the San Andreas fault for approximately 200 miles, northwest and southwest from region of Tejon Pass.

1872, March 26, Owens Valley shock, most violent along the east base of the Sierra Nevada from Owens Lake to Mono Lake. This shock was exceedingly violent in a relatively small area, and as a moderate earthquake was one of the most widespread recorded in the state—felt from San Diego to Redding with horizontal movement up to about 18 ft at one point on the fault.

1906, April 18, San Francisco Bay Region, destructive from San Jose to Santa Rosa, with visible movement on the San Andreas Fault for 200 miles from Point Arena in Mendocino County to San Juan in San Benito County. The horizontal displacement was up to 21 ft and earthquake damage was extensive and severe.

Other great shocks might be listed as follows:

1769, July 28, Southern California, San Gabriel Mission near Los Angeles.

1812, in fall of year, San Juan Capistrano in southern California. A church was destroyed with loss of 40 lives. The mission church at Santa Ynez 170 miles from San Juan Capistrano was completely destroyed and some lives lost. It was severe at San Gabriel with damage to church and buildings.

1863, October 21, Hayward shock, San Francisco Bay area, most violent near Hayward, where every building was damaged and many wrecked. Damage done in San Francisco was considerable.

1892, February 23, San Diego County, with many strong after shocks.

1915, October 2, Pleasant Valley, Nev., about 40 miles south of Winnemucca, a very severe shock in a nearly uninhabited area. The fault was visible for about 22 miles, with vertical displacement up to 15 ft.

1932, December 20, Nevada, north of Tonopah and east of Mina, with faulting visible over an area 9 miles wide and nearly 40 miles long. There were few inhabitants or structures in the region of its epicenter, but it was felt over a large area.

Other very strong shocks but not listed as great by seismologists, from 1836 to 1933, number 22 and have occurred up and down the state of California.

But it was the earthquake of March 10, 1933, which rang the bell as far as California was concerned. The lesson of 1906 seemed to have been forgotten. Earthquakes were taking place at times most convenient to the public so that despite extensive property damage, great losses of life comparable to those in other countries had been avoided.

With an aroused public the State Legislature passed the Field Act for school buildings and the Riley Act for all buildings. The registration laws already in effect for civil engineers were amended to separate another group of specialists in design—the structural engineers. From that day to this, every structural engineer and architect in the state of California has had to acquaint himself with the problem of design for resistance to seismic shock as well as the building codes and laws pertaining to such design. In the foreground of this campaign for more adequate construction, has been the Division of Architecture. Backed by the Legislature, the old Appendix A and the present Title 21, have been the weapon used by the Division in its enforcement. It is an act written by engineers, for engineers, and carried out by engineers.

One of the men who has come up the road in the effort to carry on this program of enforcement of sound engineering principles in school building construction and is today one of the top men in the Division of Architecture, is the chairman of this session, Ernst Maag. Mr. Maag not only has been active with the Division of Architecture, but has worked diligently with all building codes. He has also put a lot of his own time and energy in research problems pertaining to structural engineering. He has worked with others very extensively on developing standards and codes of practice now in general use by ASTM and other agencies.

BUILDING DESIGN FOR LATERAL FORCES

By Henry J. Degenkolb[1]

The building public is rapidly becoming conscious of the necessity of designing structures for lateral forces. On the Pacific Coast we have experienced major destructive earthquakes and the presumption is that they will reoccur. The Midwest has recently experienced several cyclones and the East and South have suffered hurricanes. The deficiency of certain types of structures under these adverse forces has been graphically demonstrated to the public.

While there has been considerable study and discussion of the forces involved and the reactions of large structures to these various phenomena, the problems of the small building have been almost entirely overlooked in engineering literature. As a matter of fact, engineers and architects in most parts of the country even today do not consider lateral loading on small structures to be an engineering problem and many codes completely ignore it. In the past few years, considerable interest has developed in shock loadings from atomic or other high energy bombs. This, together with recent hurricanes and the continuing interest of Pacific Coast structural engineers in the earthquake problem, has led to the inauguration of several research studies and has given impetus to many discussions on the engineering design problems involved.

One of the primary tools that the structural engineer has to resist these lateral forces in the smaller structures is the use of the roof or floor framing as a diaphragm. California engineers have used diaphragms for over 25 yr, and yet almost none of the basic principles or details of design have appeared in engineering literature. Most of our procedures have been learned by word of mouth and by discussions among small groups, and as a result many misconceptions, prejudices, and superstitions have arisen and have even appeared in our building codes.

What is a diaphragm, how is it designed, and how does it act? What are the basic principles involved? Instead of trying to evolve some wordy description and becoming involved in the problems of semantics, let us try to answer these questions by using an example of a simple structure.

In Fig. 1, we have a simple, boxlike one-story structure. Let us consider the lateral forces acting thereon and the necessary design to resist them. First, consider the sidewalls, with wind pressure on the windward side or suction on the leeward side. In seismic action, the inertia loads act similarly to wind or the shock waves of an explosion. The wall framing spans from foundation to the roof as a beam, and must be designed as such in addition to the column action due to the vertical loads in the roof. The top of the wall thereby produces horizontal loads against the roof framing. The wall must be attached to the roof framing strongly enough to resist the forces involved.

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[1] Partner, John J. Gould & H. J. Degenkolb, Consulting Engineers, San Francisco, Calif.

The roof framing now acts as a large girder taking the reactions of the walls as horizontal loads and spanning to the end walls which must resist these reactions. The roof system that carries these loads is called the diaphragm. In some cases it is actually framed with diagonal rods or angles to form a bracing truss, which is the only case generally considered in engineering literature. From observation, a few tests, and considerable engineering intuition, the custom has developed of using the actual roof framing, that is, the wood or plywood sheathing, concrete or metal deck or other construction materials to serve as the structural element to resist these lateral loads. How is this done?

Considering this roof diaphragm as a girder with the external loads shown, we see that there are the usual shear and moment forces to resist. Generally, chords are provided to resist the moment stresses. The roof material itself resists the shear stresses. This sounds simple, and in some materials such as with concrete or plywood, it is. The engineer can use ordinary engineering methods of design and arrive at acceptable results. When using wood sheathing or the usual steel decking, the action is considerably more complicated and extensive tests are necessary to determine the internal method of resistance and to prove the strength of the assembly.

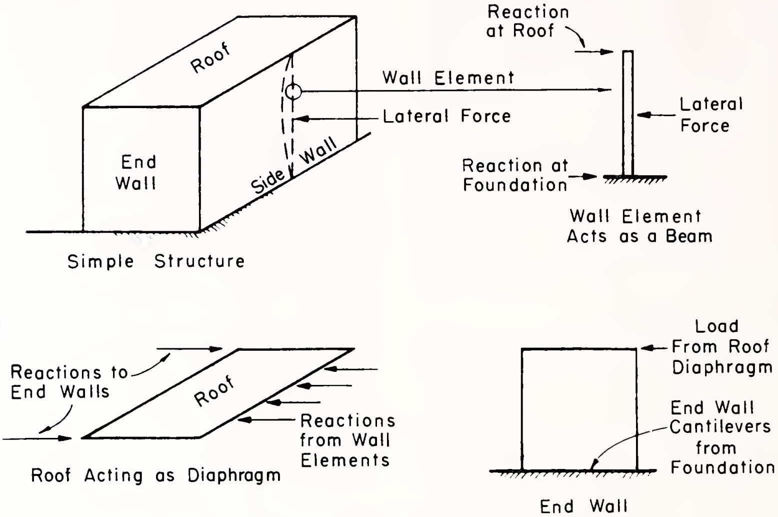


FIG. 1.—Lateral Forces on a Simple Structure.

At this stage of the problem let us summarize our diaphragm problem as we see our requiremets so far. Naturally, since it is serving as part of the structure, it should be strong enough to resist the normal vertical loads, it must not deteriorate with time or weather, and it must be reasonably economical in its construction. The diaphragm must have some reserve of strength so that it can act after normal deflections have taken place. For example, in Fig. 2, when reasonable foundation settlement takes place or truss deflection occurs, its value should be relatively unimpaired. The assembly as a whole must be capable of some degree of analysis in order to predict its strength, and its strength must be proven by test. And the diaphragm must be able to resist deflection.

As seen in Fig. 3, the loads on the diaphragm will cause it to deflect. This deflection allows the walls, in turn, to deflect. If the wall is stiff, is short, is brittle, or is fixed at the base and the diaphragm deflects too much, the wall can be cracked.

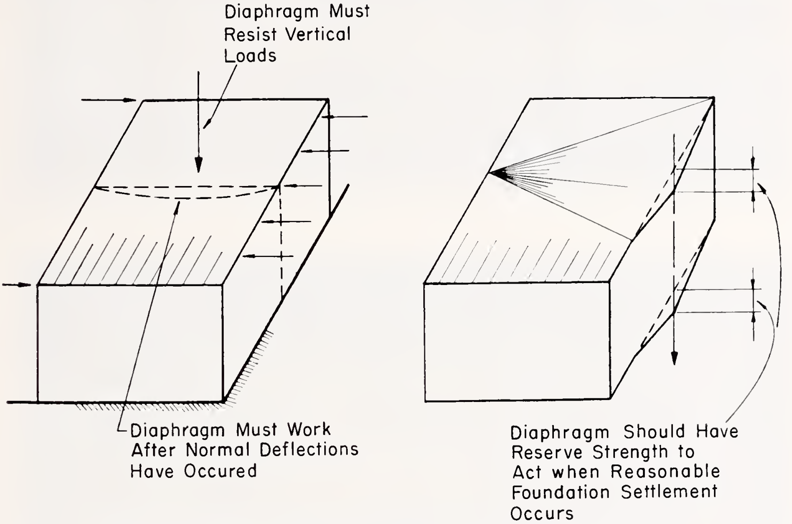


Fig. 2.—Certain Conditions Effecting Diaphragm Resistance.

By assuming the wall to be fixed at the base, it can be shown that

$$\Delta = \frac{96H^2f}{Ed} \qquad \text{(1)}$$

where:

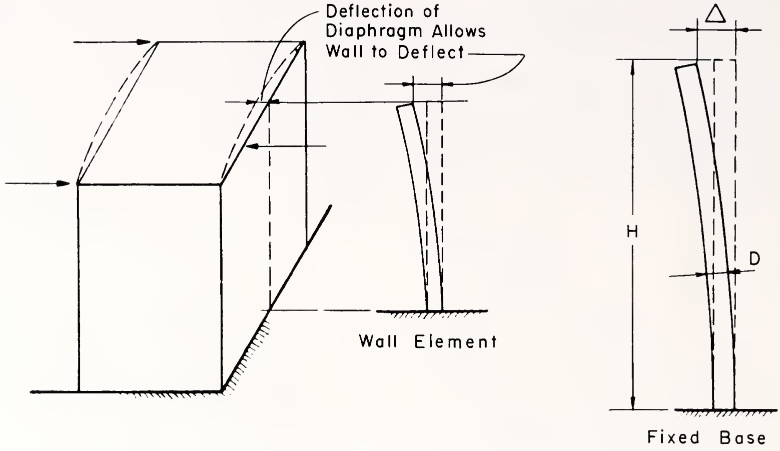
Δ = the deflection,
f = fibre stress,
E = bending modulus of elasticity,
H = height of wall, ft., and
d = thickness, in.

A group of Southern California engineers have proposed that diaphragm deflections be limited to

$$\Delta = \frac{75H^2f}{Ed} \qquad \text{(2)}$$

where f has been increased by 33 per cent above normal working stresses as is customary for lateral forces. It can be seen that for fixed conditions this is a rather conservative approach since we have the factor of safety of the difference between the normal working stress and the cracking stress. In a condition where the wall is fixed as by a loading dock in Fig. 4, or by construction in a story below, this or a similar formula may be very useful in determining the limiting deflections. However, no stretch of the imagination could indicate any validity in the other cases shown such as in a wood stud wall where rotation takes place in the plates and not bending of the studs, or in a masonry wall where the foundation construction does not fix the base. As a result larger deflections might safely be allowed than indicated by the above formula.

Now, as in most engineering problems, we have been assuming that our loads are known. For wind forces, the maximum design forces certainly are known. And in this case, at least, the stiffest possible diaphragm is probably the best, all other factors being equal. If the earthquake factors in our building codes are correct and realistic in all cases, the same assumption should also apply, but is that assumption true? Are our earthquake forces known? From the 1952 Kern County earthquakes we have several examples of seeming contradictions. In Fig. 5 we have a rigid shear wall in a high school designed for present code forces which has failed. In Fig. 6, hollow tile partitions in the Tehachapi Prison for women were badly shattered although at uniform building code stresses



$$\underset{\text{(Inches)}}{\Delta} = \frac{96H^2 f}{ED}$$

H = Height in Feet
f = Fiber Stress
E = Bending Modulus of Elasticity
D ~ In Inches

FIG. 3.—Effect of Diaphragm Deflection on Walls and Columns.

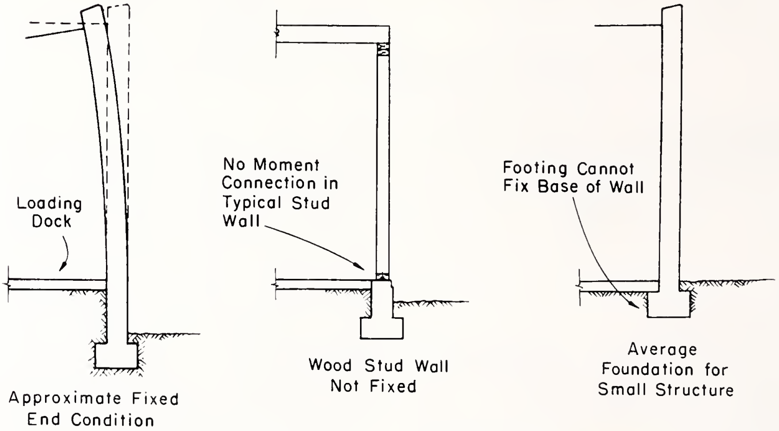
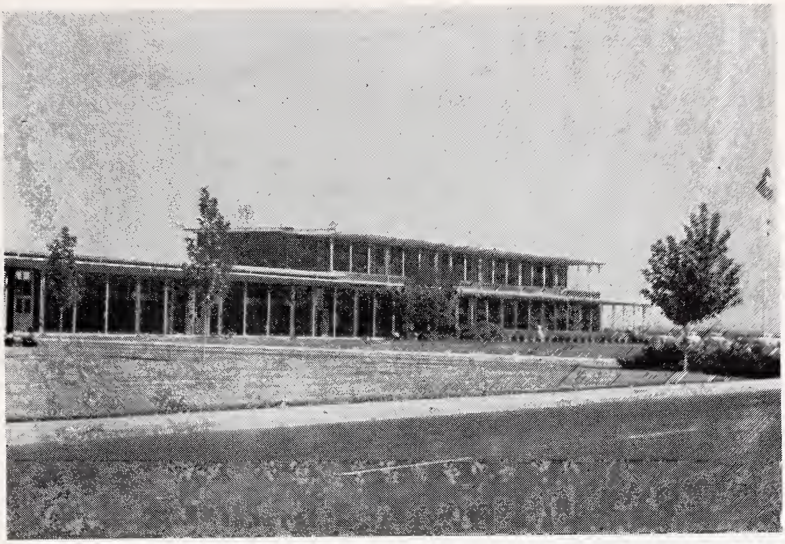
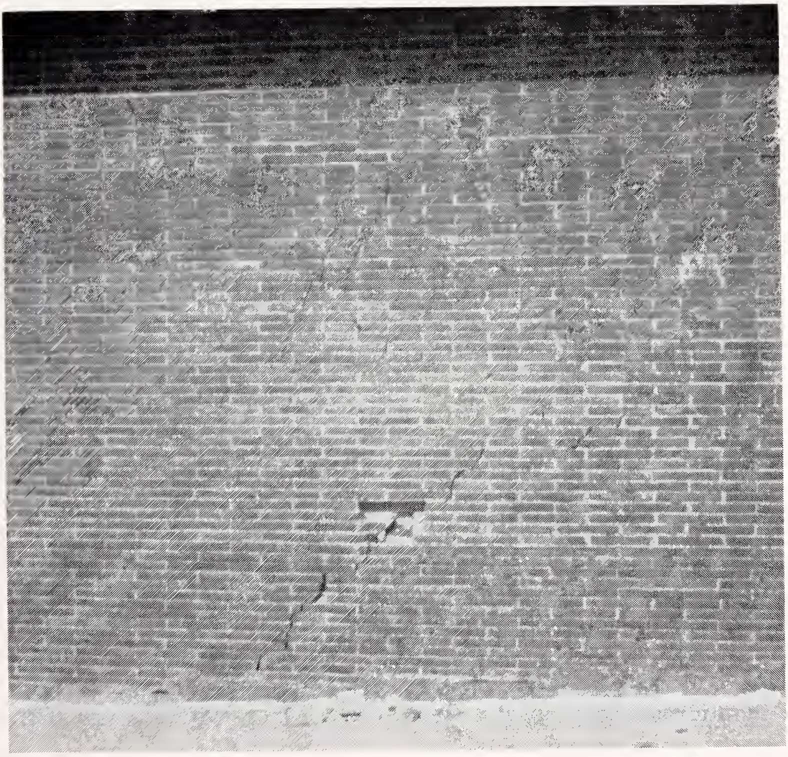


FIG. 4.—Various Wall Conditions That may be Affected by Diaphragm Deflections.



(*a*) The Second Story Crosswall Near Center of Picture was Cracked by the Earthquake.



(*b*) Close-up of Wall. This Cracking Occurred at Very Low Calculated Shear Stresses.

FIG. 5.— Arvin High School Earthquake Damage.

they will almost "figure" in shear for 13 per cent G lateral load. In Fig. 7, ordinary masonry construction in Tehachapi failed completely. Yet the unbraced board and batten building shown in Fig. 8 was undamaged, and the unbraced corrugated warehouses shown in Figs. 9 and 10 were undamaged. The unbraced warehouse shown in Fig. 10 was considered to be so safe that it was used as a Red Cross emergency station after the earthquake. Why this big discrepancy? Why should water tank details fail at stresses that indicate loads of 25 per cent G and over when these and many other unbraced buildings show no damage whatsoever?

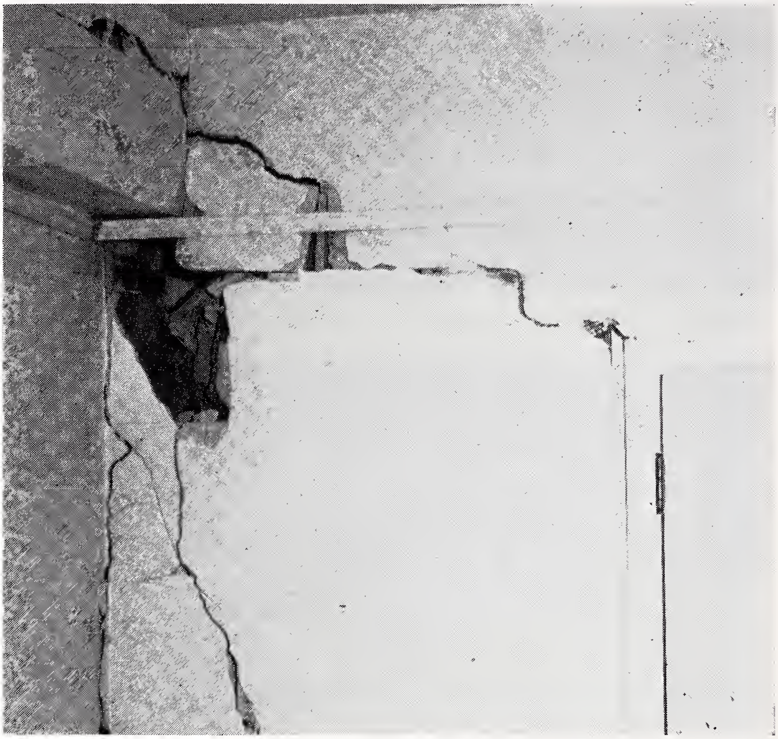


FIG. 6.—Hollow Tile Partitions in Buildings at the Tehachapi Prison for Women. These Partitions Very Closely Spaced, at Uniform Building Code Stresses, Will Almost "Figure" in Shear for 13 per Cent G Lateral Load.

Obviously, from the examples I have just cited, there is, at least, one common denominator that separates the damaged from the undamaged construction, and that is the degree of flexibility. But where is flexibility recognized in our codes? How do we design the desired "flexible" diaphragm when we have just proven that we want rigidity and not flexibility? To gain some insight into the problem, at least, even if not the solution, let us review some of our engineering fundamentals.

In evaluating the design of any structure there are two general basic approaches. The first of these is the trial and error one—the proof of time. In

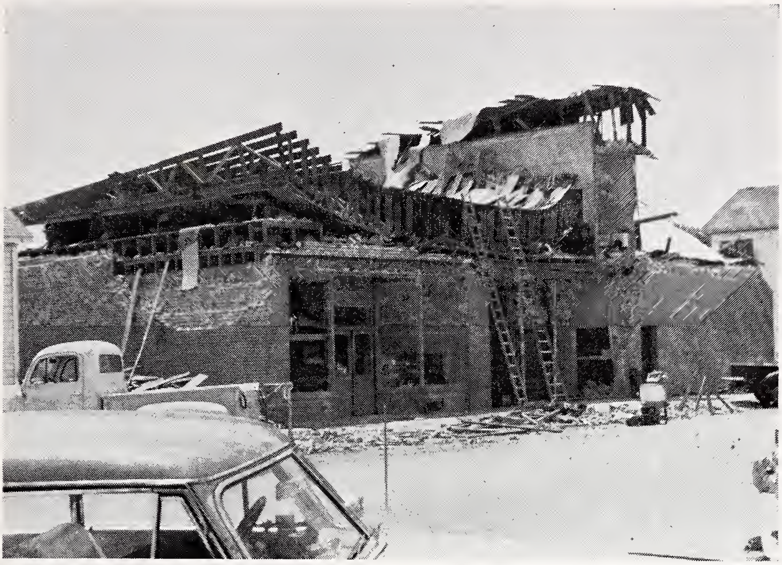


FIG. 7.—Lodge Hall in Tehachapi. The Roof Over the Second Floor Auditorium now Rests Upon the Piano and Chairs After the Walls Collapsed.

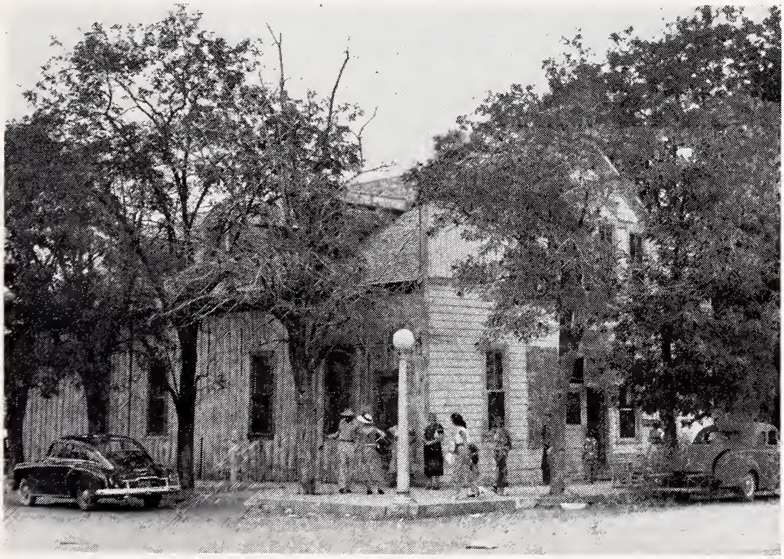


FIG. 8.—Clark Hotel, Tehachapi. This old Board and Batten Wood Structure, Several Inches out of Plumb and With no Calculable Bracing was Undamaged by the 1952 Earthquakes.

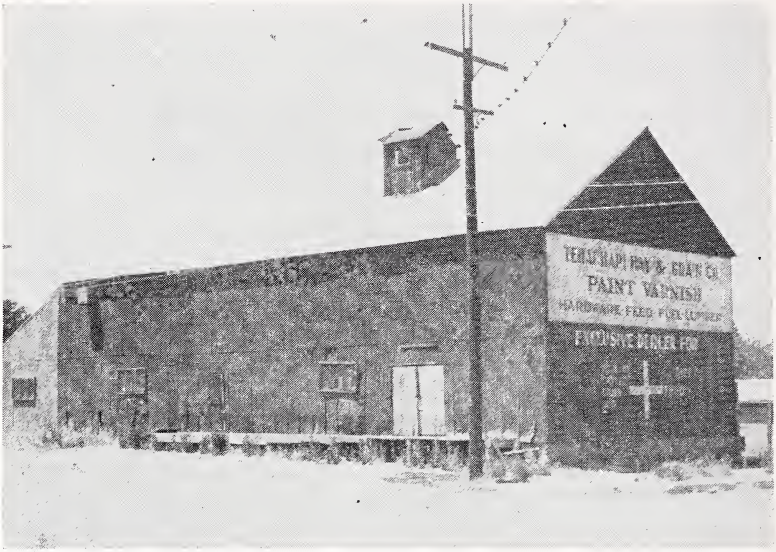


FIG. 9—Corrugated Iron Warehouse in Tehachapi. This Structure has a Timber Frame Without Bracing and was Undamaged.

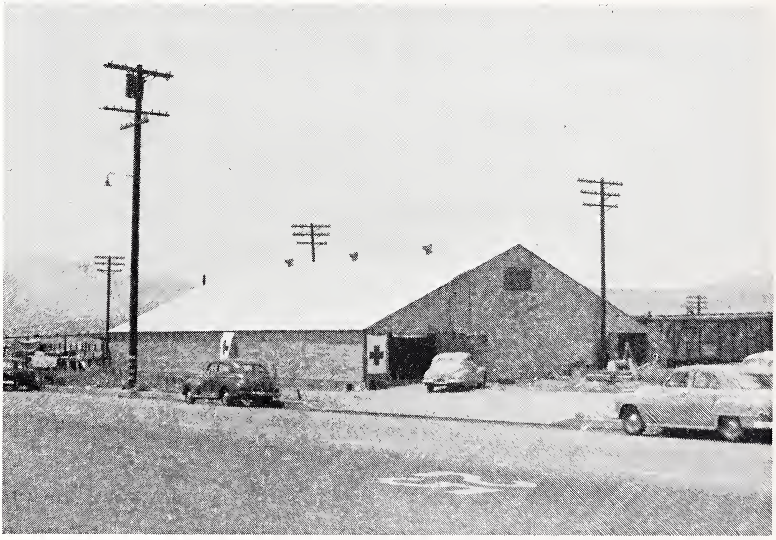


FIG. 10.—Unbraced Timber and Corrugated Iron Warehouse in Tehachapi. This Structure was Unharmed by the Earthquake.

using this approach, if it is necessary to span a certain distance with a wood joist floor, a certain size joist is tried. If it falls down or deflects too much, a larger size is used and the process repeated until a satisfactory floor is obtained.

Long before we had our formulae for bending moments, maximum stress, etc., builders were constructing bridges, buildings, towers, domes, and other structures. If one fell down, that was unfortunate; they built the next one differently or heavier, and eventually one was built that stood up. Experience was gained and empirical rules of thumb were developed and some major structures resulted. The longest timber bridge ever built, a 390-ft span, in the 1760's, was built this way. So was Louis Wernwag's "Colossus Bridge" over the Schuylkill River in Philadelphia. This timber bridge, built in 1812, spanned 340 ft successfully until it burned down in 1838. These structures are even more remarkable when one realizes that Squire Whipple's developments on truss analyses were not presented until 1847. The domes of the old cathedrals were built in this manner, and many are still standing today after centuries of use. It is not a method to be dismissed lightly, as it formed the foundation for our engineering profession.

The second approach to structural engineering problems consists of determining the forces that exist and by applying certain laws of stress and the ability of a material to resist that stress, determine mathematically what structure is needed to resist that force with an adequate factor of safety. Instead of just applying a load to a floor joist to see if it will stand up, we now calculate a bending moment and by using an appropriate fiber stress in a formula based on Hooke's law, we determine the proper size of joist that will safely carry the required load.

When all factors, relationships, and data are known, the second method, of necessity, must agree with the first elementary method. When we do know all factors, we can closely predict the results that can be obtained only expensively and laboriously by tests.

We engineers are mathematically inclined by nature or we would not have survived the type of training we were given. So we appreciate and endeavor to use this second method. While this second method is certainly valid and appropriate when all factors are known, it can certainly be misleading if certain portions of the data are not known. And yet, from our basic inclination, at times we endeavor to apply the second method before all of the data are available.

Now, just how do these basic approaches affect our lateral force design?

At the present time our lateral force design is based on the first method. Coefficients are based on observation and past experience. The requirements as to parapets, fastening of walls and ornaments, foundation ties and general attitude of design are all based upon observations of structures that either survived a destructive earthquake or were demolished by it.

In recent years, a large amount of effort is being expended in gathering data for applying the second method to the art (not science) of seismic-resistant design. The U. S. Coast and Geodetic Survey has developed, installed, and serviced instruments to measure basic earthquake motion, and several good records are now available. Various investigators have studied the application of these data to idealized simple structures of various types. Studies have been made of how the findings on these idealized structures may compare with actual prototype structures. The motion records of the U. S. Coast and Geodetic Survey indicate accelerations of the order of magnitude of 30 or 40 per cent or even more. By applying the spectrum response of idealized structures to various earthquake records, Housner has found that this has been amplified through resonance to accelerations of the order of 500 per cent. Studies of internal damping and some foundation effects have reduced this theoretical maximum again to the order of about 100 per cent of gravity. However, studies to date do not lower these theoretical figures appreciably, and there is increasing pressure that our code values should be increased to a level more closely approaching this theoretical figure.

In the face of these developments, history tells us that the Nippon Kogyo Bank in Japan, a rigid reinforced-concrete building designed by Naito for $6\frac{1}{2}$ per cent G, was undamaged in the 1923 Tokyo earthquake.

We have observed that a major San Francisco office building successfully withstood the 1906 earthquake with minor damage. Recent calculations indicate that it can be "figured" to have withstood about 0.4 per cent G corresponding to about 10 lb per sq ft wind. We see numerous instances of steel or timber buildings having negligible resistance to lateral forces coming through major earthquakes with absolutely no damage at all.

Certainly, to date, the results obtained by the two approaches to earthquake design do not agree. Based only upon observations, a rather small coefficient is adequate for many types of structures. And our theoretical second approach has not developed to the point where these small coefficients can be explained in usable mathematical terms.

Recent earthquakes in Kern County and Eureka, Calif., have given us various examples of this fact, but these earthquakes have also pointed out some seeming contradictions. Neither of these earthquakes could be considered major, and there are numerous instances in both locations where structures with negligible lateral force resistance suffered little or no damage. On the other hand, structures were damaged which indicated that there was an equivalent accelerating force of at least 25 per cent G. Some of these structures were designed according to presently accepted code requirements. And here we have the basic paradox that troubles a designer. Why does the same earthquake damage certain structures even when they have a certain inherent lateral resistance, and leave many others untouched, some of which theoretically have very little resistance? Until our engineering methods can adequately explain such discrepancies, we shall basically have to rely on the first primitive method of design.

Considering now the specific problem of diaphragms, the designer is faced with a dilemma. First of all, the damaging effects of an earthquake is not known as a "force" that can be commonly applied to all structures. In other words, the magnitude of the force is not known. And, secondly, it appears that a little flexibility aids a structure to survive a major earthquake. In fact, the loads may vary inversely with some function of the flexibility. We have seen instances where well-designed buildings with rigid elements, such as in Fig. 5, have failed. They have failed in such a manner as to indicate that it required a large force to cause failure. Permitting the diaphragm to deflect as in Fig. 3 relieves the load on the diaphragm because of the flexibility. And we have numerous instances of where this flexibility must have actually permitted buildings with very little lateral resistance to successfully withstand rather large earthquakes. So it is quite probable that some flexibility is desirable, but how much is the question. The flexibility also permits a secondary action with regard to walls and parapets, probably increasing the amount of strength and attachment necessary.

At the present time, no solution to the problem can be presented, but eventually it may take some form as a graduated set of coefficients. For example, if the structure has little or no deflection, the coefficient may be 25 per cent G while if it deflects 1 unit the coefficient may drop to 15 per cent, and if it can safely deflect 2 units the coefficient might be only 5 per cent G. The present earthquake code of Japan accomplishes a similar result in another way. Whereas a unit masonry structure must be designed for 20 per cent G, a more flexible timber or steel structure can be designed for only 60 per cent as much or 12 per cent G.

A more immediate observation can be made, however, concerning our methods of testing and evaluating our diaphragms. Our present methods of considering the effects of static loads and putting a premium on the least flexible may be wrong. For the present, we should, at least, remember, in our evaluation how the diaphragm fails. Is it still coherent and resisting load or has the failure been brittle? In overloads, what reserve of strength has it? Have the benefits of flexibility as well as the disadvantages been accounted for?

As a classic example of how far wrong we are going, tests made as early as 1929 have indicated that ordinary plaster walls will resist about 900 lb per linear ft shear, but engineers have been reluctant to use them as major shear resisting elements because of their proven lack of dependability and their tendency to crack badly. And yet, present codes permit a much weaker element with a much more brittle performance, sheet rock, to be used at values above that of diagonal sheathing. Certainly in this case we are overlooking the experiences of the past and are using empirical code-permitted values of force to justify unsuitable materials.

Conclusions

From the discussion above, I believe that the following general conclusions may be drawn:

1. Diaphragms are useful, economical tools in designing structures to resist lateral loads;
2. Diaphragms must serve to economically resist vertical loads, without deterioration through exposure, and must have sufficient reserve strength to function adequately in spite of secondary stresses caused by normal deflections, foundation settlement, and other circumstances;
3. When loads are known, such as wind forces, diaphragms should be as rigid as possible;
4. Under seismic and shock loadings some flexibility or at least toughness of a structure is desirable, based on observations of past performances;
5. The art of theoretical analysis has not as yet developed to the point where mathematical predictions of structural behavior will agree with the actual behavior in major earthquakes, and
6. Our present lateral force coefficients and design practices are generally based on observations of structural behavior.

Many of the old buildings used rather flexible diaphragms. New materials should have basically similar mechanical properties as those used in the past in order to use the same coefficients. If the mechanical properties of new materials are substantially different, other and possibly larger coefficients may be necessary to assure safe construction.

TESTS OF CANTILEVERED WOOD MULLIONS FIXED IN DIAGONALLY SHEATHED AND PLYWOOD PANELS

By Charles Peterson[1]

The current practice of providing unobstructed uniform natural lighting in our public schools has led to the extensive use of long continuous glass areas sometimes extending the full length of the classrooms. This leaves very little if any wall area remaining to provide the necessary strength and rigidity to the building along its longitudinal axis.

In order to minimize replacement costs, and the physical hazards resulting from broken glass in the event of a strong earthquake or high wind, it is important that the deflections in such a structure be kept small. This is not an easy task when wall areas that normally could be utilized to act as rigid shear panels are not available.

One solution to this problem that has been widely adopted is to provide continuous lines of mullions or columns, uniformly spaced in the walls to suit standard sash sizes. To assure a rigid cantilever action, the mullions are securely anchored into the sheathed wall areas below the window openings.

It was the purpose of the Oregon tests to determine the degree of fixity that these shallow sheathed wall sections impart to the mullions, so that the deflection or distortion in the plane of the window openings could be predicted and thus held within reasonable limits.

The investigation included both static and vibratory tests, the latter for the purpose of ascertaining what effects the vibratory loads, simulating strong earthquake shocks, may have on the capacity of the wall panel in maintaining its initial fixity on the mullion.

The California State Department of Public Works, Division of Architecture, and the Oregon Forest Products Laboratory, entered into a cooperative agreement to build and test a series of eight wood wall sections in the spring of 1955, to obtain information that could be used in designing such structural units. Structural Engineers from the California State Division of Architecture designed the test specimens and outlined the loading procedures.

Construction and Testing

A typical classroom building designed to furnish uniform bilateral lighting is shown in Fig. 1. The construction of the test panels and the loading procedures used were intended to produce an overall distortion similar to that which would occur in such a building when subjected to longitudinal forces.

The principal structural features of the test panels are shown in Fig. 2. There were two basic panel frames constructed. One contained a 4 by 6 in. mullion and the other a 6 by 6-in. mullion, each 12 ft high.

Panels of three different sill heights (8, 6, and 4 ft), were tested in a given series. To save time and material, the 8-ft high panel was tested first in each instance. After the testing of the 8-ft high panel had been completed, the sheathing material was carefully removed from the framing, and a 2-ft section was cut off the ends of the studs to make the 6-ft high specimen. The mullions in all cases were left undisturbed. The salvaged sheathing was reused for the new specimen but was oriented to avoid using old nail holes in either the sheathing or the framing. The same procedure was used in converting the 6-ft into the 4-ft high specimens.

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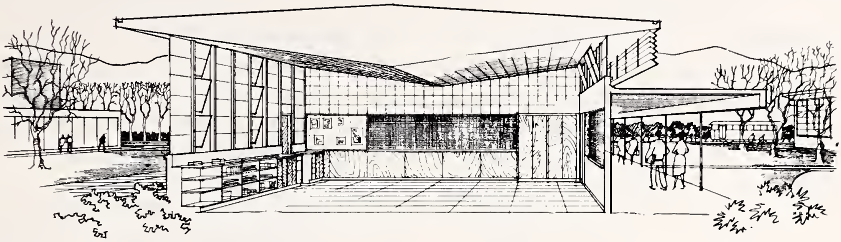


FIG. 1.—Section Typical Classroom.

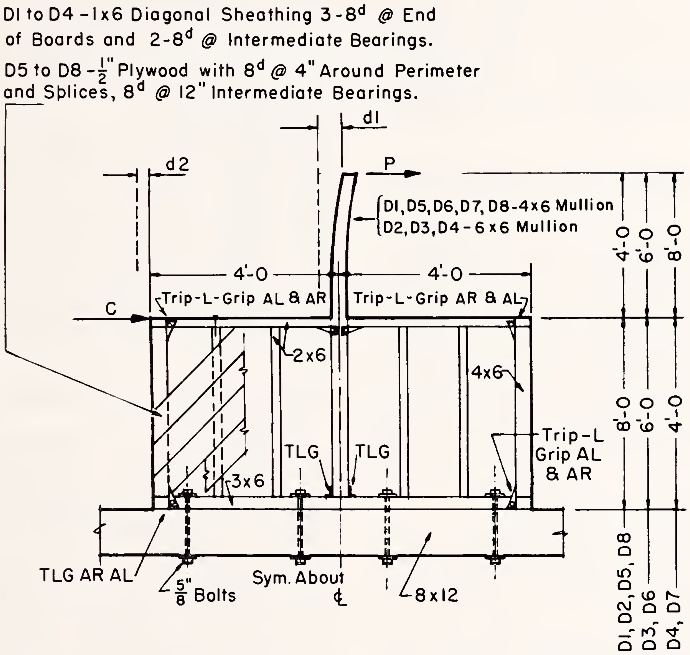

D1 to D4 - 1x6 Diagonal Sheathing 3-8^d @ End of Boards and 2-8^d @ Intermediate Bearings.
D5 to D8 - $\frac{1}{2}$" Plywood with 8^d @ 4" Around Perimeter and Splices, 8^d @ 12" Intermediate Bearings.
d1
d2
P
D1, D5, D6, D7, D8 - 4x6 Mullion
D2, D3, D4 - 6x6 Mullion
4'-0
4'-0
Trip-L-Grip AL & AR
Trip-L-Grip AR & AL
C
2x6
4x6
TLG
TLG
3x6
Trip-L Grip AL & AR
TLG AR AL
$\frac{5}{8}$" Bolts
Sym. About ℄
8x12
4'-0
6'-0
8'-0
8'-0
6'-0
4'-0
D1, D2, D5, D8
D3, D6
D4, D7

FIG. 2.—Construction Details for Wood Wall Sections D1 Through D8.

Design of the panel is intended to simulate the effect of mullions spaced 4 ft on center. Loading at C represents the reaction from a second mullion.

The framing lumber consisted of nominal 2 by 6-in. Douglas fir No. 1 grade; the lumber sheathing was 1 by 6-in. No. 1 common Douglas fir boards; the plywood sheathing was $\frac{1}{2}$-in. C-D Douglas fir Plyscord grade; the 4 by 6 in. and 6 by 6-in. mullions were of select structural grade Douglas fir. All lumber was kiln dried commercially, and had a moisture content that ranged from 14 to 20 per cent at time of construction, and from 10 to 16 per cent at time of test. Common wire nails were used throughout.

DISCUSSION

Vibration Test:

Test specimen D8 was violently shaken at the free end of the mullion by means of an eccentric cam rotating at 120 rpm, equivalent to an earthquake frequency of 0.5 seconds. It imparted an effective oscillation of ±0.95 in. to the mullion end. The vibrations were maintained for periods of 2 min and 10 min. Static load deflection readings taken before and after each vibration period indicated that the vibrations caused no appreciable change in the capacity of the panel to maintain its holding power on the mullion.

Plywood Compared with Diagonal Sheathing:

Test specimens D1 and D5 were identical in every respect except that D1 was diagonally sheathed with 1 by 6 boards and D5 was sheathed with $\frac{1}{2}$-in. plywood. Load deflection readings on the two specimens indicated that plywood and diagonal sheathed panels develop virtually the same degree of fixity on the mullion.

Effective Degree of Fixity:

When a simple cantilever is so completely restrained that no elastic or inelastic rotation can take place at its point of anchorage, it can be said that the cantilever is fully fixed. In actual practice however, this condition can only be approximated and the primary purpose of the study was to determine how nearly conventional construction approaches the ideal condition of full fixity.

In Fig. 3 is shown, at an exaggerated scale, a deflected mullion restrained by a shallow wall panel.

Deflection d1 is the observed total lateral movement of the free end of the mullion from its initial position.

Deflection d2 is the observed total movement of the panel.

Deflection d3 represents the calculated elastic deflection of the mullion assuming it to be fully fixed (ideal condition) at the window sill line.

Deflection d4 or (d1-d2) represents the effective distortion in the plane of the window openings that could cause glass breakage. This latter deflection is made up in part by:

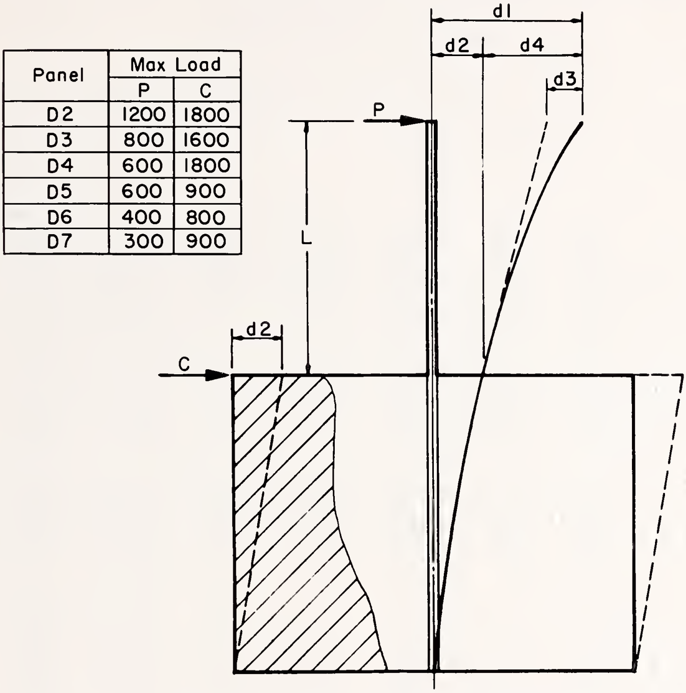
1. The leaning or tilting of the mullion due to the movement d2 of the panel;
2. Partly by a certain amount of elastic rotation of the mullion at the window sill line, and
3. The remainder by the elastic deflection d3 in the cantilevered portion of the mullion itself.

By definition, the effective degree of fixity may then be taken as the ratio d3/d4 which represents the efficiency of the panel as compared with an assumed fully fixed condition. The tests show that the degrees of fixity varied appreciably between the various test specimens but followed a fairly consistent pattern.

Factors Influencing the Degree of Fixity:

A study of the test results shows that the degrees of fixity developed by the panels ranged from 40 per cent to 70 per cent. This variation seems to be governed chiefly by the intensity of loading and the stiffness of the mullion itself. (The effective stiffness of the mullion may be represented by the ratio EI/l, where E equals the modulus of elasticity of the wood, I equals the moment of inertia of the mullion section and l equals the length of the cantilevered portion). The effects of these factors on the behavior of the test panels are shown in Table I.

Up to now there has been no general agreement as to what the allowable deflection or distortion in the plane of the window openings should be. Some code authorities believe that a deflection of $\frac{1}{16}$ in. per ft of height of window opening should be considered as the maximum limit, while others think that $\frac{1}{10}$ in. per ft of height would be a reasonable allowance without unduly increasing the danger of glass breakage. In any case, it is obvious that the size and number of the glass panels contained in the window openings would be an important factor to consider when establishing deflection limits.



Panel	Max Load	
	P	C
D2	1200	1800
D3	800	1600
D4	600	1800
D5	600	900
D6	400	800
D7	300	900

FIG. 3.—Illustration of Deflected Panel.

The maximum loads noted, stressed the 4 by 6 in. mullion to 2340 psi in bending and the 6 by 6 in. mullion to 2080 psi.

Application of Fixity Factors:

As stated before, the fixity factor (degree of fixity) is taken as the ratio d3/d4, where d3 is the calculated elastic deflection of the cantilevered portion of the mullion, and d4 is the actual or total effective deflection determined by tests.

Therefore, in order to determine the actual deflection of a mullion, the designer merely divides the calculated elastic deflection by the fixity factor. For example: if the calculated elastic deflection is 0.24 in. and the fixity factor is taken as 60 per cent for the given conditions, the actual deflection would be

$$\frac{0.24}{0.60} = 0.40 \text{ in.}$$

This latter deflection, of course, should not exceed the allowable for the window height in question.

CONCLUSIONS

Evaluation of test results within the scope of this investigation leads to the following general conclusions.

(1) The efficiency of the wall panels would not be impaired by strong, vibratory loads of 12 min duration.

(2) There appeared to be no appreciable difference between diagonal sheathing and plywood in the degree of fixity developed by the panels.

TABLE I.—DEGREE OF FIXITY.

D2, D3, D4, Diagonally Sheathed Panels: 6 by 6 mullion

$I = 70\ \overline{\text{in.}}\ 4,\ E = 1{,}740{,}000$ psi

D5, D6, D7, Plywood Sheathed Panels: 4 by 6 mullion

$I = 22\ \overline{\text{in.}}\ 4,\ E = 2{,}110{,}000$ psi

Wall Panel	Height, *l*, ft	Loads, lb		Mullion Stress, psi	*d*1, in.	*d*2, in.	*d*3, in.	*d*4, in.	Fixity, *d*3/*d*4, per cent
		P	*C*						
D2	4	240	360	410	0.18	0.01	0.07	0.17	41
		480	720	830	0.41	0.07	0.15	0.34	44
		720	1080	1250	0.64	0.15	0.22	0.49	45
		960	1440	1660	0.87	0.20	0.29	0.67	44
		1200	1800	2080	1.10	0.27	0.37	0.83	45
D3	6	160	320	410	0.30	0.02	0.16	0.28	57
		320	640	830	0.56	0.03	0.33	0.53	62
		480	960	1250	0.87	0.08	0.49	0.79	62
		640	1280	1660	1.18	0.13	0.66	1.05	63
		800	1600	2080	1.49	0.20	0.82	1.29	64
D4	8	120	360	410	0.47	0.01	0.29	0.46	63
		240	720	830	0.98	0.02	0.58	0.96	61
		360	1080	1250	1.41	0.04	0.88	1.37	64
		480	1440	1660	1.82	0.07	1.17	1.75	67
		600	1800	2080	2.27	0.09	1.46	2.18	67
D5	4	120	180	470	0.21	0.01	0.09	0.20	45
		240	360	930	0.39	0.01	0.19	0.38	50
		360	540	1400	0.60	0.03	0.28	0.57	49
		480	720	1870	0.82	0.08	0.38	0.74	52
		600	900	2340	1.01	0.11	0.47	0.90	52
D6	6	80	160	470	0.40	0.01	0.21	0.39	54
		160	320	930	0.70	0.02	0.42	0.68	62
		240	480	1400	1.06	0.04	0.63	1.02	62
		320	640	1870	1.36	0.06	0.85	1.30	65
		400	800	2340	1.68	0.07	1.06	1.61	66
D7	8	60	180	470	0.59	0.01	0.38	0.58	65
		120	360	930	1.13	0.01	0.75	1.12	67
		180	540	1400	1.65	0.02	1.13	1.63	69
		240	720	1870	2.21	0.02	1.50	2.19	69
		300	900	2340	2.71	0.03	1.88	2.68	70

(3) Reducing the relative stiffness EI/l of the mullion raises the apparent fixity factor of the panel for a given loading.

(4) To keep deflections within reasonably safe limits, stiff mullions must be employed using relatively low working stresses.

(5) Considering the variables involved, such as the moisture content of the wood, the effective modulus of elasticity of the mullions which may range from 1,500,000 psi to 2,000,000 psi (if made of Douglas Fir) and other indeterminate factors, it seems that for all practical design purposes for wall units containing 6 by 6 mullions, a fixity factor of 50 per cent may be assumed, and for 4 by 6 mullions, a fixity factor of 60 per cent would be satisfactory.

REPORT ON U. S. FOREST PRODUCTS LABORATORY TESTS OF FULL-SIZE STRUCTURAL DIAPHRAGMS[1]

By R. P. A. Johnson[2]

Surveys of damage resulting from earthquakes and hurricanes showed that wood structures performed well. As a result, engineers and architects have shown increased interest in wood structures. A demand was created for more data on the behavior of wood diaphragms so that wood structures could be better engineered. Twenty-five years ago tests of diaphragms were started to meet that demand (**1**).[3] Those tests were made on models of timber walls. Later, other tests were made on 15 one-fourth scale models of 20 by 60-ft panels by the Oregon Forest Products Laboratory (**2**). Tests of models, however, did not furnish adequate information, primarily because there is no satisfactory method for correlating the results with full-scale structures. The fastenings at joints cannot be accurately scaled down and, therefore, are not comparable to those made with members fastened with full-size nails or other fastenings.

The U. S. Forest Products Laboratory started a study of diaphragm action of full-scale hayloft floor panels several years ago. The study was made in cooperation with the Agricultural Engineering Branch of the U. S. Department of Agriculture Research Service.

The cost of material and labor limited the number of full-scale tests that could be made. Three series of tests were made. Two were on panels 6 by 16 ft and one was on panels 18 by 48 ft. The results of those tests are reported in three Forest Products Laboratory reports (**3, 4, 5**). Obviously, the limited number of tests, made on panels typical of those used in farm structures, did not furnish adequate information for use in designing other types of wood structures.

An extensive testing program on full-size panels has been conducted over the past few years at the Oregon Forest Products Laboratory, the Douglas Fir Plywood Assn., and the U. S. Forest Products Laboratory. The first two organizations have published the results of most of their work (**2, 6, 7, 8**). The Forest Products Laboratory has prepared three preliminary reports (**9, 10, 11**) on the work done by them. Two of the reports were on the diaphragm action of full-scale diagonally-sheathed roof and floor panels, and the other was on full-scale diagonally-sheathed wood wall panels. The work was done in cooperation with the State of California, Division of Architecture, and the Dept. of the Army, Corps of Engineers. The reports were prepared by D. V. Doyle of the Forest Products Laboratory, who supervised all the testing. The three reports will be combined, condensed, and summarized in a single manuscript for publication. This paper is a brief summary of the tests and the findings in those three reports.

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[1] Made in cooperation with the State of California and the Dept. of the Army, Corps of Engineers.

[2] Engineer, U. S. Forest Products Laboratory, Madison, Wis.

[3] The boldface numbers in parentheses refer to the list of references appended to this paper, see p. 30.

The reports are too detailed and too extensive to permit more than a brief description of the material, constructions tested, methods of test, and a summary of findings.

FLOOR AND ROOF PANELS

Description of Material:

All the panels were built of Douglas fir. The material was all No. 1, graded under Paragraphs 187 and 204 of Standard Grading and Dressing Rules for Douglas Fir, No. 14. The joists in eight panels were 2 by 10 in., spaced 24 in. on centers, except that center joists of panels with "V" (herringbone) sheathing were 3 by 10 in. The joists in one panel were 3 by 10 in., spaced 4 ft, and in one 4 by 10 in., spaced 6 ft on centers. The sheathing on 24-in. spaced joists was 1 by 6 in. and that on joists spaced 4 or 6 ft was 2 by 6 in.

Moisture Content:

The material was dry at the time of test. The 2-in. framing ranged from 11 to 13 per cent moisture content and that of the 1-in. sheathing from 9 to 12 per cent. To insure similar dry material in 3- and 4-in. thick joists, 3-in. joists were laminated from 2 pieces of 2-in. stock planed to equal thickness before gluing and the 4-in. joists from 3 pieces of 2-in. stock likewise planed. Their moisture content at test ranged from 10 to 12 per cent. The 2-in. sheathing averaged from 8 to 9 per cent moisture content.

The moisture content of a representative portion of the framing and sheathing was determined daily with a moisture meter. The lumber was protected during construction by waterproof canvas. When purchased, the moisture content of the framing lumber and 2 by 6-in. sheathing averaged about 20 per cent. The 1-in. sheathing averaged about 15 per cent moisture content. All the lumber showed a general decline in moisture content during the construction period. This confirmed the general belief that exposed framing usually dries out during summer construction.

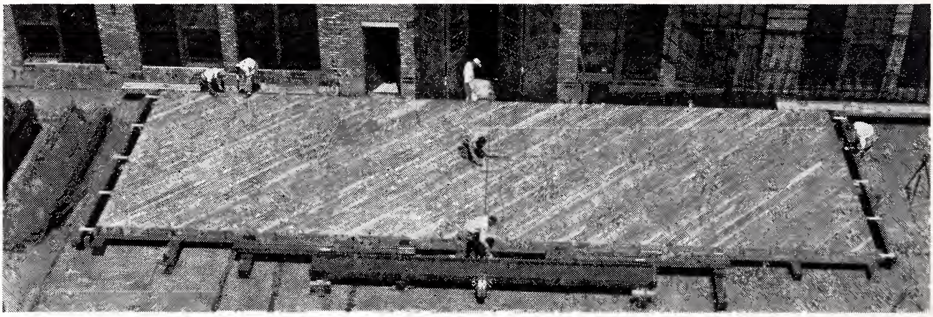


FIG. 1.—Roof Panel FA1 Being Made Ready for Test.

Steel I-beams on the far side of the panel transmit the loads at the reactions to the columns of the laboratory building. The testing machine within the laboratory building applied the load through a ¾-in. wire rope to the series of three I-beams arranged to apply the load at the fifth point of the panel. The load is measured by a weighing capsule mounted between the wire rope and the large I-beam and is recorded on a dial gage located within the building.

Description of Panels:

All panels tested were 20 by 60 ft. Their size made it necessary to test them outdoors. They were tested on a paved parking area. Figure 1 shows the setup.

The joists were continuous over a 20-ft span.

The sheathing was laid at 45 deg with the framing members and nailed at each bearing with eight penny common wire nails. Three nails were driven at the end of each sheathing board and two at each intermediate joist. To simulate service conditions resulting from shrinkage, the sheathing boards were spaced $\frac{1}{8}$ in. and a separation of $\frac{3}{64}$ in. was left between sheathing boards and framing members. Figures 2 and 3 show the construction of the panels with 2-in. joists. Figure 4 shows the construction of panels with 3- and 4-in. thick joists.

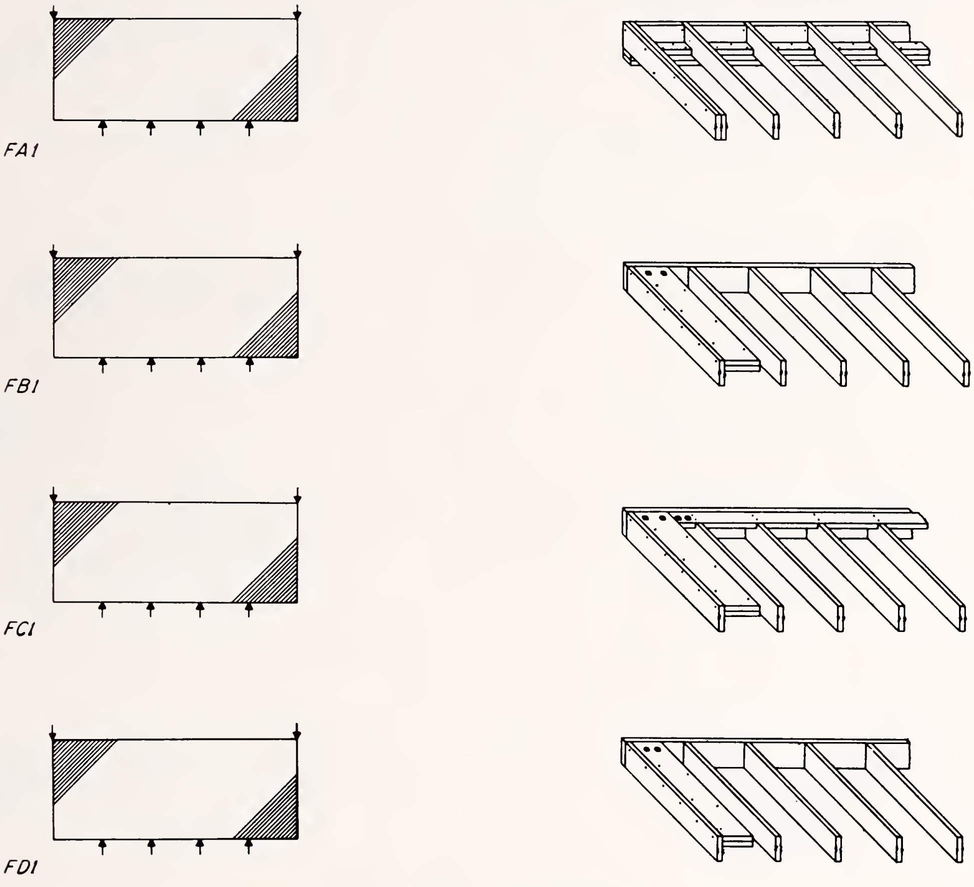


Fig. 2.—Schematic Panel Details.

Eleven tests of wood roof or floor panels were conducted to determine the strength and rigidity of as many constructions. The constructions studied included:

Panel FA-1—Conventional construction with cut-in blocking of 2 by 10-in. and 2 by 4-in. lumber at ends of joists and no stiffeners.

Panel FB-1—Continuous headers and stiffened end chords.

Panel FC-1—Continuous headers, stiffened end chords, and stiffened longitudinal chords.

Panel FD-1—Effect of short sheathing on the same framing as panel FB-1.

Panel FE-1—This panel featured V-pattern sheathing on framing with continuous headers and stiffened end chords.

Panels FF-1, 2, and 3—Three test runs were made on the one panel. The panel had conventional cut-in blocking with V-pattern sheathing. The first test was made to evaluate the stiffness. The second was made after reinforcing the corners with 4 by 10-in. solid blocking bolted to joists and sills. The third test was made after driving two additional nails at each bearing point in the outer 4 ft of the perimeter.

Panels FG-1 and 2—Two tests were made on one panel that had a continuous header, bolted splice, stiffened end chords, 4 by 10-in. joists spaced 6 ft, and 2 by 6-in. diagonal sheathing. The first test was made to a shear load of 400 lb per ft of width. In the second test 2 by 6-in. planks were nailed flatwise beneath the sheathing midway between the joists.

Panel FH-1 had continuous headers, bolted splices, stiffened end chords, 3 by 10-in. joists spaced 4 ft, and 2 by 6-in. diagonal sheathing.

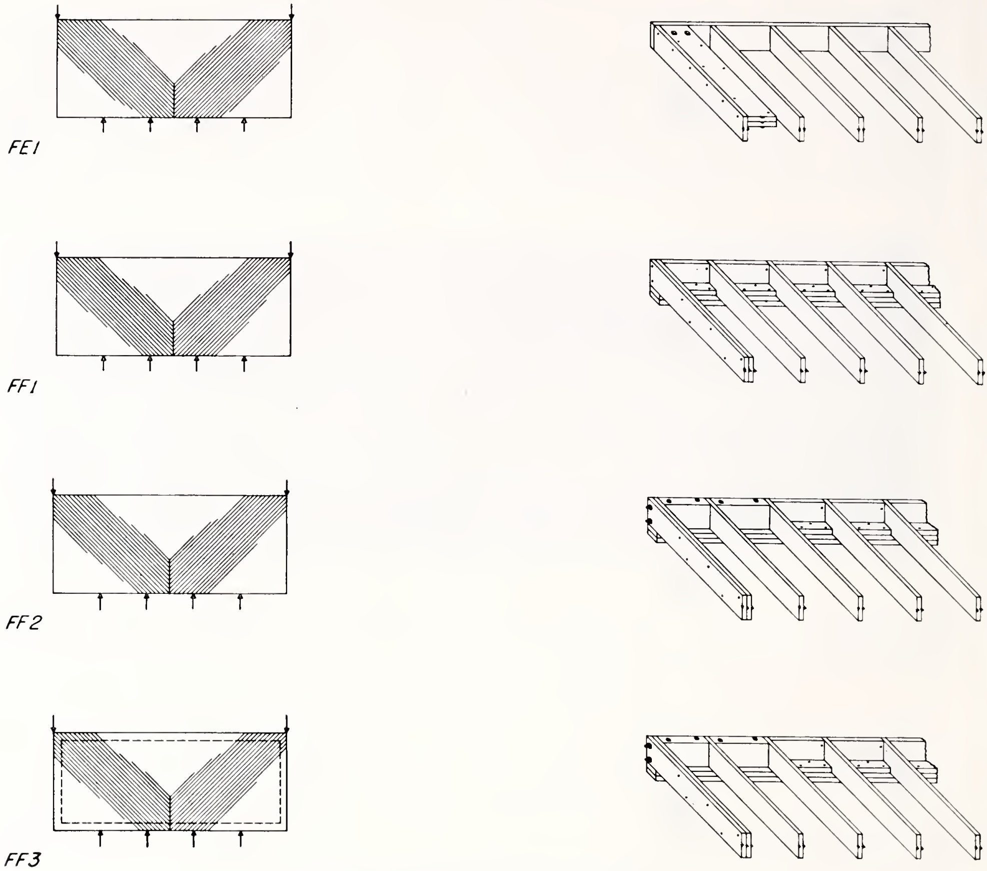


Fig. 3.—Schematic Panel Details.

Testing Procedure:

The panels were tested in bending under fifth-point loading. The load was applied by wire rope attached to the movable head of a testing machine. A hydraulic capsule, mounted between the cable and loaded I-beam, measured the load on a dial gage. The deformation of panel was measured at a number of points along its perimeter.

The loads are expressed in pounds per foot of width, that is, reaction load divided by panel width.

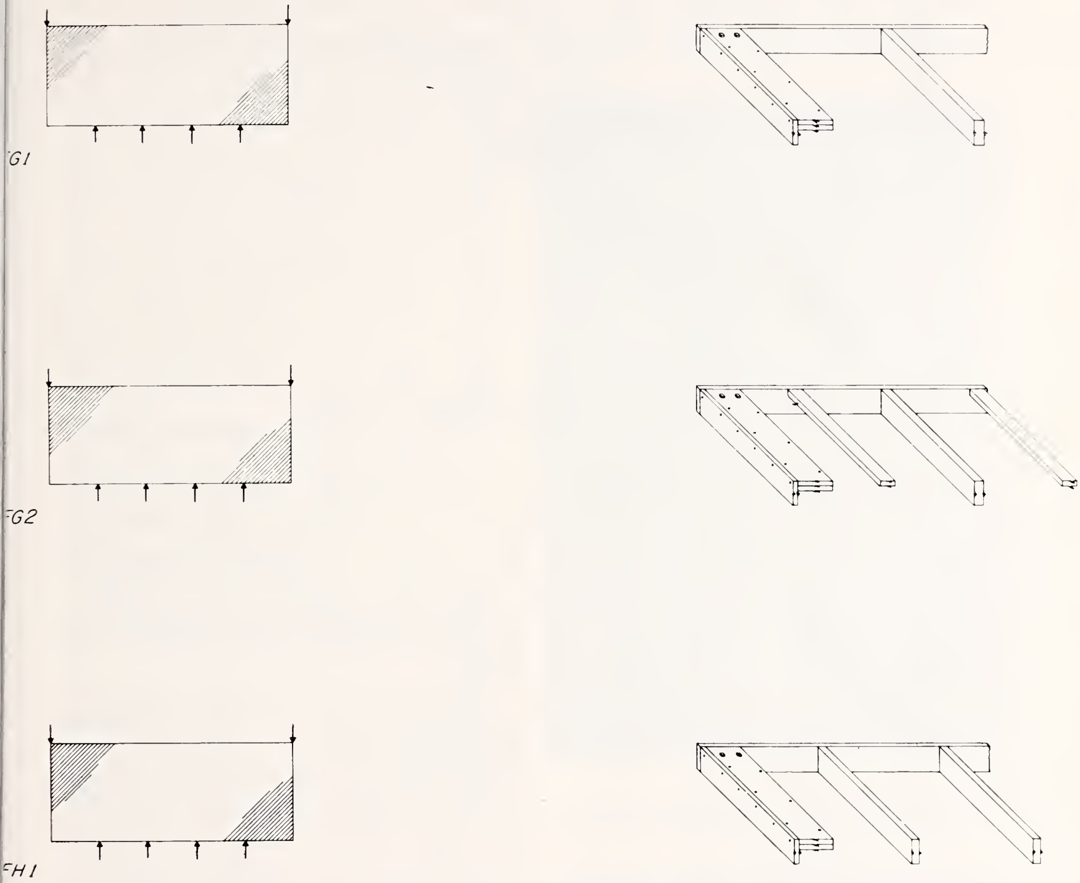


Fig. 4.—Schematic Panel Details.

An initial load of 12.5 lb per ft of panel width was placed on panels. The load was then applied at 25-lb-per-ft intervals, the deflection read, load released, and residual deflection read. Interval loading was repeated until failure or until an excessive deflection was reached. A few series of control tests were made to evaluate the properties of the material and to determine the strength of the joints.

Discussion of Results:

The results of tests are shown by load-deflection curves for the center of the longitudinal chord on the unloaded edge of the panels (Fig. 5 for panels with 2-in. joists and Fig. 6 for panels with 3- and 4-in. thick joists).

Panels with 24-in. joist spacing.—All panels with continuous headers (FC-1, FE-1, FB-1, and FD-1) were stronger and stiffer than any of the panels with let-in headers (Fig. 5). The panel with both stiffened end chords and stiffened longitudinal chords (FC-1) was stiffest and strongest. The conventional panel (FA-1) with let-in header blocks and no stiffeners was with the exception of panel FF-1 the weakest. The use of V (herringbone) in place of diagonal sheathing improved the stiffness and strength only slightly but provided symmetry in panel action and deflection. Reinforcing the corners of the conventional panel, as in panel FF-2, improved the stiffness and strength, and those properties were further improved by perimeter nailing but not enough to equal the poorest panel with continuous headers and end stiffeners. The panel with short diagonal sheathing spanning two joists (FD-1) was not so strong or so stiff as a similar panel with sheathing boards spanning five joists and emitted loud noises during loading. The increase in strength and stiffness, however, was not in proportion to the increase in the length of the boards.

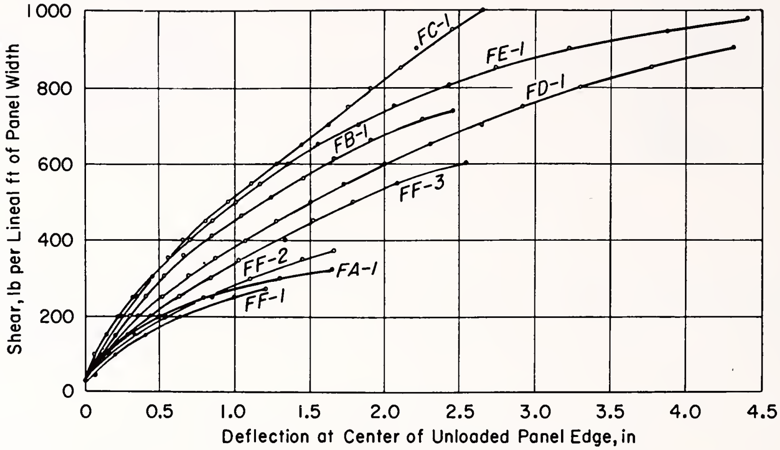


FIG. 5.—Load-Deflection Curves for Panels FA to FF, inclusive, all with 1-in. sheathing boards.

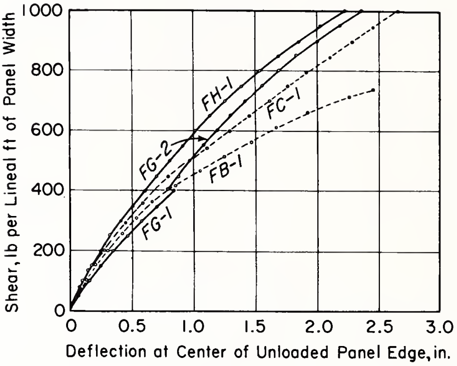


FIG. 6.—Load-Deflection Curves comparing Panels FC and FB, incorporating 1-in. sheathing boards, to panels FG and FH having 2-in. sheathing.

Panels with 4- or 6-ft Joist Spacing.—The results of tests of these two panels (FG-1 and FH-1) are compared in Fig. 6 with tests of two of the stiffened panels with 2-in. thick joists and 24-in. spacing. One panel (FC-1) has both end and longitudinal chord stiffeners; the other (FB-1) has only end chord stiffeners. Panel FH-1 with the 3 by 10-in. joists, spaced 4 ft, was the strongest and stiffest panel tested.

The panel with 4 by 10-in. joists spaced 6 ft (FG-1) was not so rigid within the limits of the tests (400 lb per ft of panel width) as a similar panel with 2-in. joists spaced 2 ft or the panel with 3-in. joists spaced 4 ft. Panel FG-1 was then stiffened by nailing 2 by 6's flatwise to the underside of the sheathing halfway between the joists. This not only made the panel stiffer than all except the FH-1 panel but reduced the excessive deflection of the sheathing between the joists when a man walked over the panel.

Wall Panels

Wall panel tests were conducted on a dual panel representing two sections of a frame wall of a building with a story height of 12 ft. Seven series of tests on wall constructions that incorporated various details of sheathing and framing were included in the testing program. The two sections of the panel were constructed of full-size members and bolted to a center beam. The sections were laterally loaded simultaneously through the center beam in the direction of their length (Fig. 7). A study of the height-length ratio effect of $1:1\frac{1}{2}$, $1:1$, and $1:\frac{1}{2}$ was made by cutting the initial 17-ft, 4-in. panel to a 12-ft panel and then to a 5-ft, 4-in. panel.

Description of Material:

All panels were built of Douglas fir. No. 1 boards were used for sheathing. The 2 by 6-in. framing members of two panels were No. 2 and those of the remaining panels were No. 1 grade. The moisture content of the framing lumber at the time of testing varied from 10 to 12 per cent and of the sheathing from 8 to 11 per cent.

Description of Panels:

Figure 8 shows the important differences in panels. In general, the panels represented the construction commonly used in frame buildings. They were framed with 2 by 6-in. studs and plates and 3 by 6-in. sills. The studs were spaced 16 in. on center. Double studs were used along the outer edges of panels. The sections were bolted together at the sills through an 8 by 8-in. timber.

All panels were reinforced at the corners with heavy angle iron corner brackets, except initial test panel WT. Solid blocking was offset and end-nailed between the studs of each panel section. The 8 by 8-in. timber extended above the panel and provided a means of applying load simultaneously to each panel. The sheathing was laid at 45 deg to the framing members and nailed at each bearing. Three nails were used at the end of each sheathing board and two at intermediate bearing points. The sheathing was not nailed to solid bridging. The sheathing boards were separated by $\frac{1}{8}$-in. openings, and a separation of $\frac{3}{64}$ in. was left between the sheathing boards and the framing members.

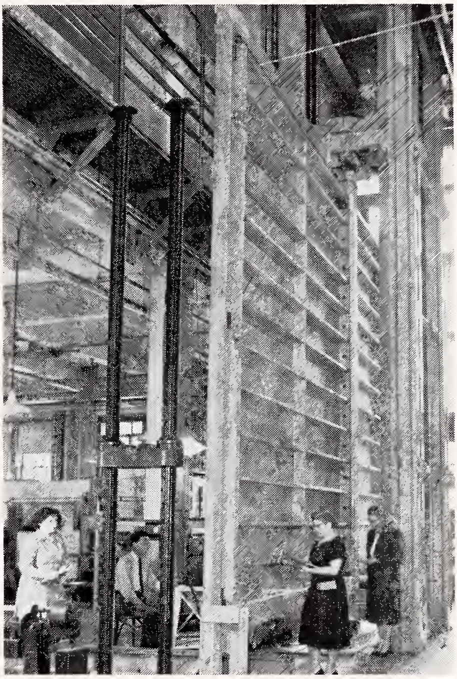
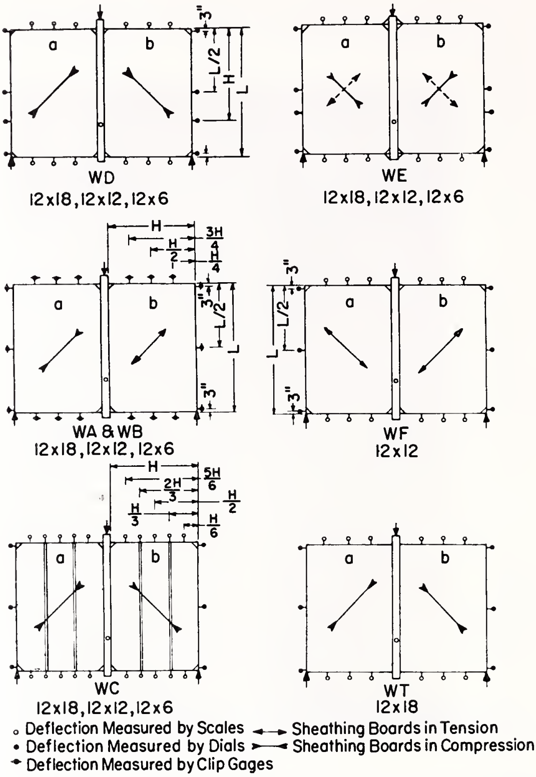


Fig. 7.—General Test Setup Showing Panel WA-18 at the Beginning of the Initial Test.

Panel WD with the boards in the panel sloping in opposite directions (herringbone pattern), placing the sheathing in compression, was used as a control. Heavy steel angle brackets were attached at all corners prior to test.

Panels WA and WB are duplicates. They are of conventional construction, but using full-length sheathing boards applied diagonally in the same direction, which placed them in tension on one section and in compression on the other. Corner brackets were placed on the loaded edge only. As a result, the compressive action of the sheathing caused the double plate to pull from the studs at the lower corner of that section whose sheathing was in compression. A corner bracket was added when the framing members had separated about 1 in.


a
b
WD
12x18, 12x12, 12x6
3"
L/2
H
L
WE
12x18, 12x12, 12x6
H
H/2
3H/4
H/4
3"
L/2
L
WA & WB
12x18, 12x12, 12x6
WF
12x12
H
2H/3
5H/6
H/2
H/3
H/6
WC
12x18, 12x12, 12x6
WT
12x18
○ Deflection Measured by Scales
• Deflection Measured by Dials
Deflection Measured by Clip Gages
Sheathing Boards in Tension
Sheathing Boards in Compression

WD—Diagonally sheathed with boards in compression.
WA—Diagonally sheathed with boards on panel *a* in compression and on panel *b* in tension. Corner angle bracket added at lower corner of framing of panel *a* prior to final test.
WB—Same as WA.
WC—Diagonally sheathed with boards in compression. Framing reinforced with let-in 1 by 6-in. boards. Trip-L-Grip framing anchors added at ends of studs prior to final test.
WE—Diagonally sheathed on both faces of panel with boards in compression on one face and tension on reverse face. Dotted lines indicate direction of sheathing on back side.
WF—Diagonally sheathed with boards in tension. Test included only a 12 by 12-ft panel.
WT-Same as WD except that Trip-L-Grip framing anchors were attached to the framing at the ends of all studs. The corner angle brackets were omitted during the initial test. Test included only a 12 by 18-ft panel.
The locations at which deflections of the panels were determined are shown by the open and closed circles.

FIG. 8.—Schematic Details of Panels.

The two sections of frame walls, bolted together through a center post and with opposite ends resting on solid supports, were tested simultaneously by applying load to the center post. Distinguishing details of panels:

Panel WC was similar to the control, with herringbone sheathing in compression and brackets at all four corners. In addition, two 1 by 6-in. boards were let in at the third points to act as horizontal stays. After the distortion of the panels became appreciable but before failure, Trip-L-Grip fasteners were attached to the unsheathed edge of the ends of all intermediate studs.

Panel WE was the same construction as the control panel but was diagonally sheathed on both faces, with boards on one face in compression and the other in tension. Corner angle brackets were placed on opposite sides of the double studs at the center post, and single angle brackets at other corners.

Panel WF was the same as the control panel, WD, except that the sheathing of the herringbone pattern sloped down toward the center post, putting it in tension instead of compression. Only one 12 by 12-ft panel was tested.

Panel WT was the same as control panel WD, with the herringbone sheathing in compression, except that two Trip-L-Grip fasteners, attached at each end of all studs, were substituted for the corner angle brackets. The corner angle brackets were attached, however, when the 3 by 6-in. sill started to split. Only one 18 by 18-ft panel was tested.

Discussion of Results:

The deformations at the center of the dual panels are shown in Fig. 9.

The conventional method of nailing panel framing permits large distortions of the panel at low loads in single-sheathed panels with the sheathing acting in compression. Angle brackets bolted at the corners of single-sheathed panels shift failure from the end framing member to the nailed sheathing (panels WA, WB, and WD).

Let-in stays or ties applied normal to the studs add restraint and stiffness to single diagonally-sheathed panels with the sheathing acting in compression (panel WC).

Trip-L-Grip framing anchors resist joint slip in single-sheathed panels when

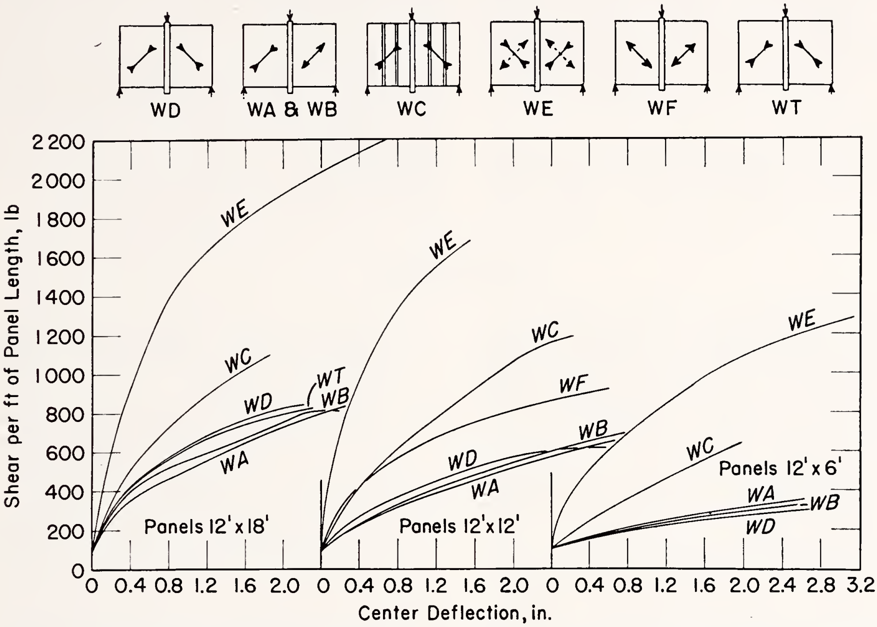


Fig. 9.—Curves Showing the Relationship of the Shear per Foot of Panel Length to the Deflection for all Panels.

The curves for panels WA, WB, WC, and WT are composite curves made up of two or more test runs. The schematic drawings above the curves illustrate the distinguishing characteristics of the panels.

the sheathing is in compression. They are more effective in longer panels and when attached to the sheathed face of the framing (panel WT).

Panel WF with diagonal sheathing in tension is stiffer than panel WD with sheathing in compression. See Fig. 9.

Panels WE with diagonal sheathing on both sides of the studs normal to one another gave shear values per foot of length two to four times that of panels with single sheathing acting in compression.

The shear resistance per foot of panel length for single-sheathed panels increases with the increase in length of the panel.

The results of these tests do not furnish data that can be used to calculate the strength or stiffness of panels of different sizes or different constructions. They do, however, show the efficiency of different types of construction and methods of reinforcing that can be used to stiffen and strengthen panels. The data, together with the accumulated data from other diaphragm tests, enable engineers to design wood structures that will better resist the force imposed by earthquakes and hurricanes and thus reduce hazard to life and property.

REFERENCES

(1) Lydik S. Jacobsen, "Experimental Study of Dynamic Behavior of Models of Timber Walls Subjected to Impulsive Horizontal Vibrations," Stanford University *Bulletin of the Seismological Society of America*, Sept. 1930.

(2) G. H. Atherton and J. W. Johnson, "Diagonally Sheathed Wood Diaphragms," *Report No. T-2*, Oregon Forest Products Lab., Corvallis, Ore. (1951).

(3) D. V. Doyle "A Study of the Diaphragm Action of 6 by 16-Foot Hayloft Floor Panels," Forest Products Lab. (1952).

(4) D. V. Doyle and J. A. Scholten, "A Study of the Diaphragm Action of 18 by 48-Foot Hayloft Floor Panels," Forest Products Lab. (1952).

(5) D. V. Doyle, "The Stiffness of 6 by 16-Foot Hayloft Floor Panels Sheathed Longitudinally Over Spaced Diagonal Subflooring," Forest Products Lab. (1956).

(6) J. R. Stillinger, J. W. Johnson, and J. L. Overholser, "Lateral Tests on Full-Scale Lumber-Sheathed Diaphragms," *Report No. T-3*, Oregon Forest Products Lab., Corvallis, Ore. (1952).

(7) J. R. Stillinger, "Lateral Tests on Full-Scale Lumber-Sheathed Roof Diaphragms," *Report No. T-6*, Oregon Forest Products Lab., Corvallis, Ore. (1953).

(8) David Countryman, "Lateral Tests on Plywood-Sheathed Diaphragms," *Laboratory Report No. 55*, Douglas Fir Plywood Assn. (1952).

(9) D. V. Doyle, "Diaphragm Action of Full-Scale Diagonally Sheathed Wood Roof or Floor Panels," Forest Products Lab. (1955).

(10) D. V. Doyle, "Diaphragm Action of Full-Scale Diagonally Sheathed Wall Panels," Forest Products Lab. (1956).

(11) D. V. Doyle, "Diaphragm Action of Wood Roof or Floor Panels Diagonally Sheathed with 2 by 6 Lumber," Forest Products Lab. (1956).

GENERAL DISCUSSION

Mr. Lyman W. Wood.[1]—Most of the tests that have been reported here this morning used static loading; I would like to ask Mr. Degenkolb what he thinks about the need for dynamic loading in tests of these lateral diaphragms?

Mr. H. J. Degenkolb (*author*).—I think it is very desirable to subject some panels to shock or dynamic loading. The testing of a large diaphragm takes a full day and sometimes more. As far as earthquake forces are concerned, a short duration of load test may indicate that much more load could safely be applied in design, similar to the 100 per cent allowable impact factor now permitted in timber bridge design.

Mr. E. George Stern.[2]—The number of feasible variations in the design of structural wood diaphragms is large and only few could be subjected to tests. An unexplored variable of major influence on the stiffness and rigidity of wood diaphragms covers the selection of the nails used in fastening the lumber sheathing to the structural framing.

All the diaphragms tested were assembled with low-carbon-steel, plain-shank, common wire nails which bend during transmission of relatively small diaphragm loads. For a number of years, (1) bright and (2) hardened (heat-treated and tempered), high-carbon-steel nails have been in mass production. These nails can be considerably stiffer than common wire nails. Consequently, their use can result in high rigidity of the joints of sheathing and framing. Since the rigidity of wood diaphragms is directly influenced by the rigidity of these joints, the wood diaphragms are more effective whenever the stiffer nails are used.

With the introduction of automatically fed nailing machines in the building field, it became feasible and practical to use a larger number of smaller nails, since automatic handling of the larger number of smaller nails can be more economical. Such smaller nails have to provide, of course, sufficient holding power to prevent their withdrawal under load application. For this reason, such smaller nails must be provided with properly threaded shanks.

When a large number of such stiff, threaded nails is used for fastening sheathing to framing, it should be possible to utilize the full strength of the lumber to transmit lateral diaphragm loads.

The results of an investigation of diaphragms assembled with such large numbers of small, stiff, and properly threaded nails might be interesting and valuable.

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[1] U. S. Forest Products Lab., Madison, Wis.

[2] Research Professor, Virginia Polytechnic Institute, Wood Construction, Blacksburg, Va.

SYMPOSIUM ON DEVELOPMENTS IN GLUED-LAMINATED AND OTHER WOOD CONSTRUCTIONS

INTRODUCTION

By L. J. Markwardt[1]

The ready availability and ease of fabrication of wood set the pattern for extensive use by early craftsmen, and there had accumulated over the years a large background of empirical information on its physical properties, characteristics, and methods of application. While wood is thus among the oldest structural materials, it significantly maintains an important place in our modern economy, as a result of the extensive engineering research and development since the turn of the century.

The parade of technical progress has appropriately included an extensive evaluation of the mechanical and physical properties of the various species of timber employing unified methods of test established as ASTM Standard D143 – 52, Standard Methods of Testing Small Clear Specimens of Timber; extensive development of technical information on factors affecting the strength, covering such factors as moisture content, rate of loading, duration of stress, fatigue, temperature, direction of grain, position of growth rings, influence of inherent characteristics such as knots, cross grain, checks, and shakes; timely development of structural grades for timber in which knots and other characteristics are limited in size and position so as to permit definite quality classes susceptible to close evaluation of strength; the establishment of working stresses for timber based on a more intimate knowledge of factors affecting strength and design; the development of improved joints and fastenings including metal connectors that have greatly widened the horizon of timber construction for large structural applications; the development of commercial and structural plywood involving the fabrication of large flat sheets; and the development of structural fiberboards.

In addition, much attention in recent years has been given to the further development of glued-laminated, stressed-skin, and sandwich constructions. Significantly, the utility and field of application of these constructions is greatly broadened by the developments in synthetic resin adhesives that permit bonds with any desired degree of moisture or water resistance. For example, glued-laminated members are now used for keels and other structural parts of ships.

From the time of its introduction, glued-laminated construction met with favor, and its acceptance and utility have resulted in the establishment of a new fabricating industry. Broadly, the term "glued-laminated construction" refers to members or structural units glued up from smaller pieces of wood, either in straight or curved form, with the grain of all the laminations essentially parallel to the length of the member. In one form of this construction, glued-laminated arches have afforded a means of achieving large clear spans in buildings with excellent architectural effects. In recent designs, spans upwards of 250 ft have been employed.

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[1] Assistant Director, U. S. Forest Products Laboratory, Madison, Wis.; Chairman, Committee D-7.

Further studies on plywood, as another versatile wood product dependent on adhesives, have resulted in broadened structural applications. It is particularly effective for facings in sandwich construction and for gusset plates in the newer forms of glued wood trusses, as well as in connector construction. At the same time there has been continued development in engineered timber construction with solid wood.

Because of the developments in timber construction in the West and the extensive use and applications in this region, the Second Pacific Coast Area Meeting of the ASTM afforded a particularly appropriate occasion to review this progress through a series of symposium papers. These presentations give an overall picture of the importance of these developments in wood construction and some details of the nature, variety, and extent of their current use and application.

DEVELOPMENTS IN STRUCTURAL GLUED LAMINATED CONSTRUCTION

By Robert E. Eby[1]

By definition, structural glued laminated members comprise an assembly of wood laminations in which the grain of all laminations is approximately parallel longitudinally and the laminations are bonded with adhesives. In addition, all laminations must be stress graded, production methods must meet exacting standards, and the adhesive used must develop a bond strength equal or greater than the strength of the wood—the basic assumption in all structural glue laminating. Laminating, as discussed in this paper, will be with reference to the above definition and requirements only, and will not include cross bonding of laminations such as plywood or veneering.

History of Laminated Construction

A short history of the structural laminating industry requires a review of the past half century, with only the last 25 yr being applicable to laminating in the United States. Except for very limited production, the birth of the industry in this country should be established as 1934, as it was then that U. S. Forest Products Laboratory initiated studies to produce data and determine strength properties of glued laminated structural members. The initial research program dealt primarily with the effect of such factors as curvature, end joints, defects, and methods of gluing pressure application. A 46-ft wide building using glued laminated arches was constructed that year at Madison, Wis. to assist in these studies. In 1936, a member of the Forest Products Laboratory staff inspected a wide variety of structures in European installations, some of which had been in service for as long as 30 yr. The culmination of this basic research was the printing in 1939 of *Technical Bulletin 691*, "The Glued Laminated Wooden Arch." The industry, up to this point, was mainly producing small sections for farm buildings and similar uses, but with the availability of sound technical data and the demand created by the advent of World War II, the industry expanded rapidly into heavy commercial and industrial fields.

This extensive expansion of the laminating industry in the late 1930's was tied in very closely with another major development—the availability of the synthetic resin water-resistant glues. Prior to this time, practically all structural laminating in this country and in Europe had been with casein adhesives—a *moisture*- but not *water*-resistant glue. Closely following the availability of water-resistant glue was the development of proper techniques of mixing, spreading, and curing the glues, and preparing the lumber within the close tolerances required for surfacing, scarfing, and moisture content limits. Extensive data collected during the past 20 yr has resulted in industry "know-how" that permits the assurance of optimum results for any, including the most demanding condition of use.

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[1] Chief Engineer, Rilco Laminated Products, St. Paul, Minn.

Perhaps the prime example of structural glued laminated members meeting the most severe conditions of use is the outstanding performance established during and since World War II of laminated keels, stems, and frames in minesweepers and other Navy craft. The shortage of white oak for marine use led to the development of laminated replacements, with a resultant increase in strength, stiffness and durability compared to the mechanical joined counterparts. This and similar outstanding service records of laminated construction in interior and exterior service have largely overcome the early reluctance of many engineers to depend on glue for the structural integrity of the members.

The next important phase in the development of the laminating industry was new studies undertaken by the Forest Products Laboratory during World War II to supplement the limited data previously developed. These studies included more extensive tests on the effects of end joints and growth characteristics, as well as studies of full size beams and columns and continuation of the development of adhesives and techniques in gluing of both treated and untreated lumber. Knot studies were made on the structural grades used by the industry to establish the probability of occurrence of knots and their effect on strength. The result of these studies was the establishment of realistic working stresses enabling the industry to use more economically the raw material, and in 1951 the Douglas Fir, Southern Pine, and Hardwood Laminating Specifications were developed as a result of the combined research and development programs of the glue manufacturers, the regional lumber associations, the Forest Products Laboratory, Bureau of Ships, and other governmental agencies in cooperation with the laminating industry. In 1954, the Forest Products Laboratory combined the latest technical data available in preparing *Bulletin 1069*, "Fabrication and Design of Glued Laminated Wood Structural Members." (1)

Research and development have not slacked off, and further tests have established such criteria as the acceptance of the fatigue resistance of glued laminated members and in setting requirements on nailing in lieu of clamping for pressure. Glue manufacturers continue to improve adhesives, and improved methods of production are constantly being developed by the industry.

The outstanding development in the past five years was the birth, in 1952, of the American Institute of Timber Construction (AITC) as a national, non-profit service organization of the engineered timber construction industry. The AITC has been instrumental in uniting the industry and is leading the way in research and technical developments related to engineered timber construction. The AITC "Timber Construction Standards" (2) is a basic guide for engineers, architects, and other specifiers and users of wood, and covers the latest technical data and industry practices including design, fabrication, erection, and a code of standard practice. The AITC "Inspection Manual" (3) outlines the equipment and procedures required to produce acceptable quality glued laminated structural members. A construction manual is partially completed and will include recommendations and standard practices covering such much needed subjects as trusses and bracing, camber and deflection, gluing of treated and treating of glued lumber, arch design, short form specifications, permissible checking, and other subjects which have not previously been covered in the manner required for use by architects and engineers. The Standards Committee of the AITC is charged with the development of these industry standards and is composed of industry representatives, guided and assisted by advisers representing regional lumber associations, glue manufacturers, treating companies, Forest Products Laboratory, and other government agencies, as well as practicing architects, engineers, and college professors.

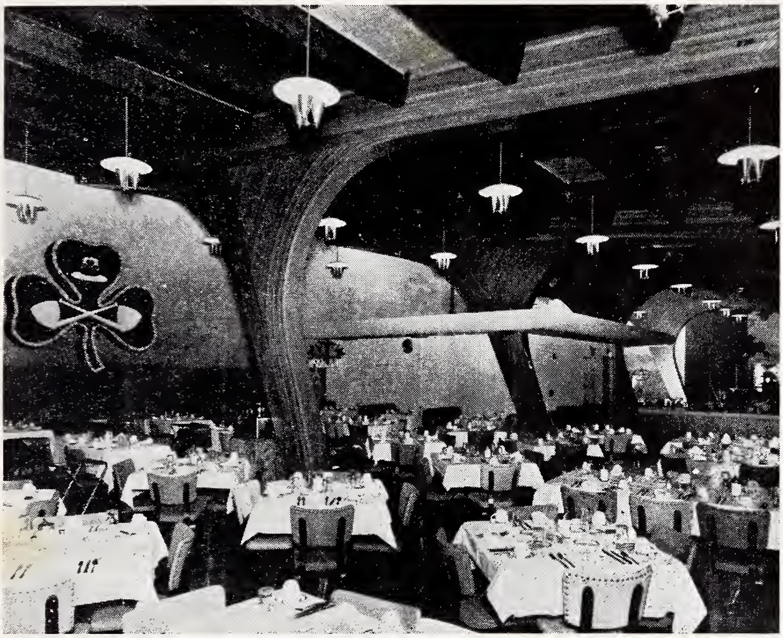


FIG. 1.—Curved Shapes Constructed with 6 Rilco Glued Laminated Frames Spaced 20 ft on Centers. Span: 43 ft.

GLUED-LAMINATED CONSTRUCTION TODAY

The many advantages of glued laminated construction, including increased strength, essentially unlimited size, architectural effects possible with curved shapes (Fig. 1) and freedom from severe checking, has created a demand for the products that has required rapid expansion of production facilities to keep pace with the needs of the construction industry. The unlimited variations and flexibility of design have been a challenge to architects and engineers which they have eagerly accepted, and each day sees new examples and applications of glued laminated structural members. Recent outstanding projects include a 247-ft clear span arch in Florida serving as a jai-a-lai court, and a 300-ft clear span arch-dome in Montana. A 250-ft clear span arch is in production, and probably the longest single timber ever produced was recently shipped for service in a bridge in Manitoba, Canada. The two timber girders were each $12\frac{1}{2}$ in. by $71\frac{1}{2}$ in. by 103 ft-0 in. long and were designed for H 15–S12-44 highway loading.

Each contained 12,000 ft board measure of Douglas fir structural lumber, 503 lb of waterproof glue, and weighed over 11 tons. The girders were pressure treated after gluing to meet the exterior use conditions of the job. Glued chord bowstring trusses in spans well over 200

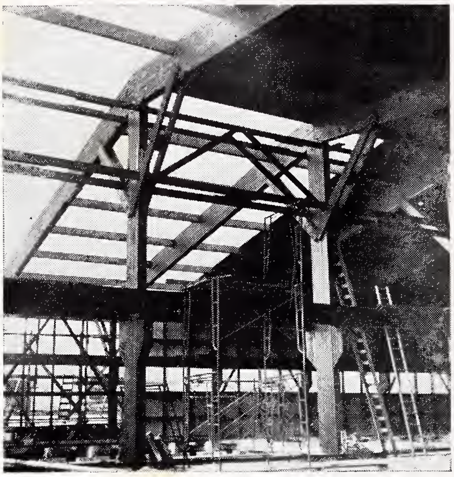


FIG. 2.—Laminated Beams and Columns in Construction of Chlorine Plant, Longview, Wash.

ft and rigid frame designs using laminated members have become commonplace (Fig. 2). In addition to the more common use of structural glued laminated arches (Fig. 3), beams, purlins, trusses, and barn rafters, laminated members are being produced for such specialty items as bleacher seats, diving boards, pike poles, cross arms, ladder rails, and other industrial uses. Use of glued laminated stringers for railroad and highway bridges is increasing, and region 6 of U. S. Forest Service, which covers Washington and Oregon, are designing and constructing 40 to 60 glued laminated stringer and truss bridges a year (Fig. 4).

The principal species used in the

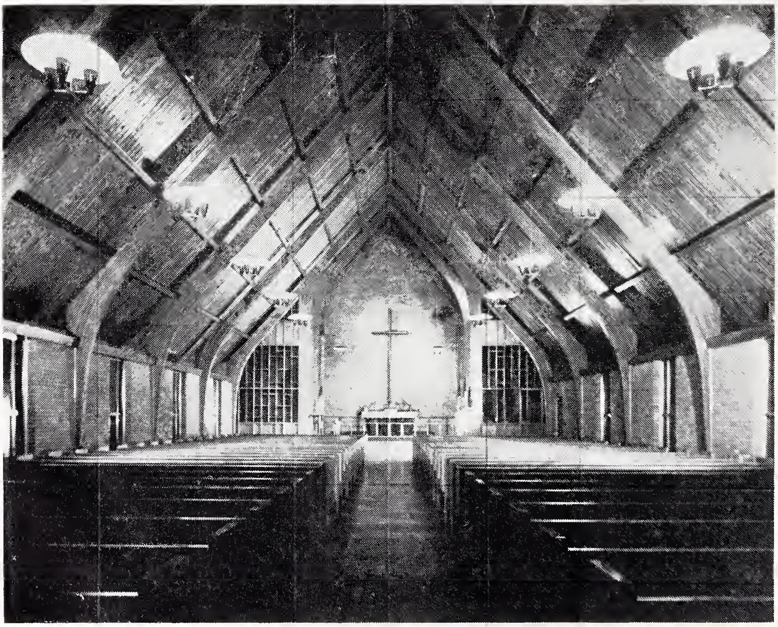


FIG. 3.—Trinity Evangelical Lutheran Church, St. Paul, Minn.
Span of Arches: 43 ft 3 in.; Height; 35 ft 3⅝ in.
Spaced: 14 ft on centers; Chancel Arches: 24 ft span spaced: 8 ft on center.

laminating industry are Coast Type Douglas fir and southern pine. Hardwoods are used mainly in ship building and specialty applications. There is some increase in the use of West Coast hemlock, inland Douglas fir, redwood, and white fir, and the future will no doubt see increased uses of these and other species.

The principal adhesives used by the laminating industry today are casein with mold inhibitor for interior use, and resorcinol and phenol-resorcinol resins for exterior use. Melamine and fortified melamine and urea adhesives are also used to some extent. The adhesives picture is by no means static, as the industry is continuously carrying on research to develop new and better adhesives and methods of application. The adhesive manufacturers hold the key to future improved processes that will result in reduced costs of the laminated product, and it is predicted there will be startling developments in the near future in methods of applying adhesives and in clamping for pressure.

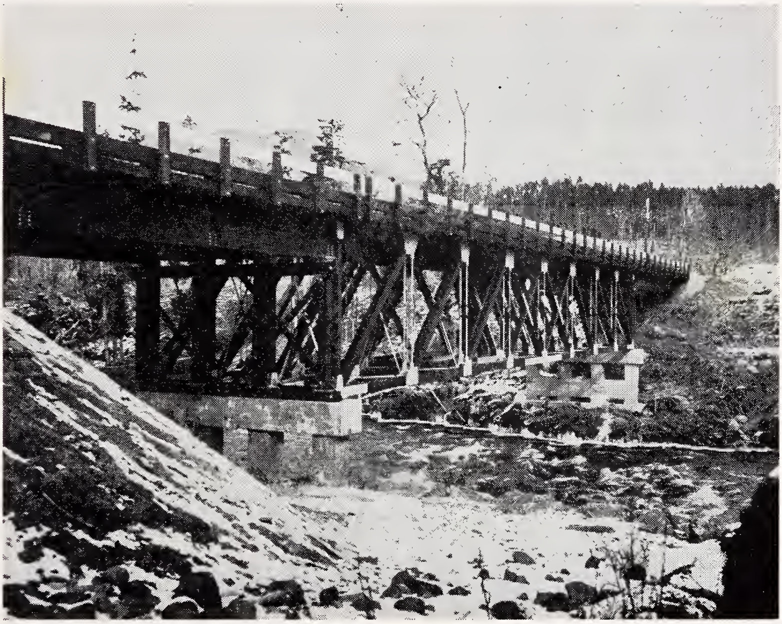


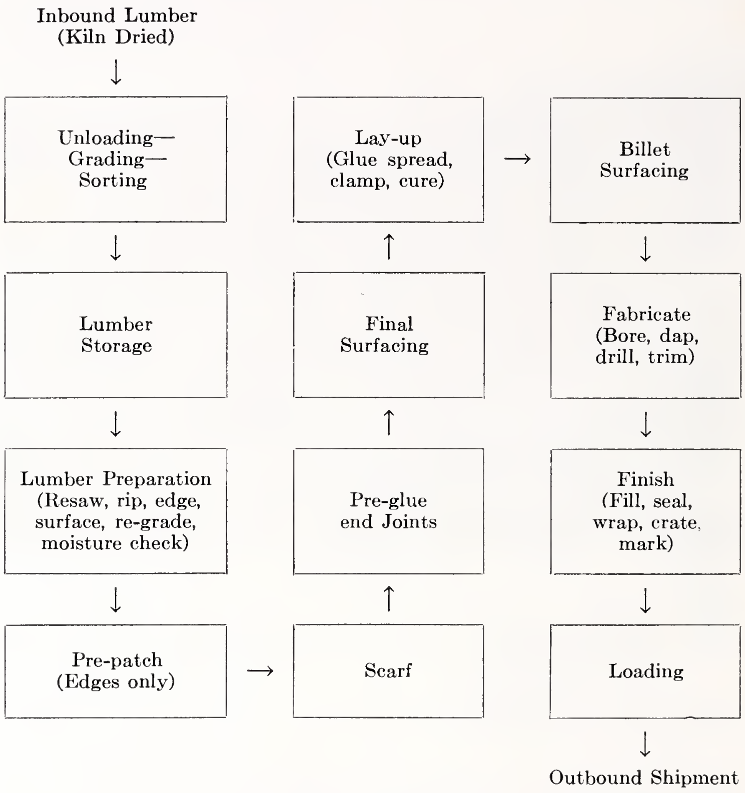
Fig. 4.—Huffaker Bridge near Randall, Washington. Constructed with Rilco Glued Laminated Wood Stringers.

Main span truss is 120 ft long, the approach on each end of bridge span 50 ft. Bridge is designed to carry 54 tons. Upper chords of the main span is 11 in. thick and from 16¼ in. at the ends to 21¼ in. at the center section.

A Modern Laminating Plant

The following is a quick trip through a modern laminating plant utilizing the latest equipment and techniques to produce a quality product at the most economical cost. Specially selected stress-graded lumber, which has been blanked to size and custom kiln-dried to a uniform moisture content in the 10 to 14 per cent range, arrives at the lumber preparation department. Here the lumber may be remanufactured to width and thickness to meet job requirements and then regraded. In the surfacing operation, an automatic moisture detector will mark any pieces not within the prescribed limits and these pieces must be returned for further seasoning.

LUMBER FLOW DIAGRAM IN A MODERN LAMINATING PLANT


Inbound Lumber (Kiln Dried)
Unloading— Grading— Sorting
Lumber Storage
Lumber Preparation (Resaw, rip, edge, surface, re-grade, moisture check)
Pre-patch (Edges only)
Scarf
Pre-glue end Joints
Final Surfacing
Lay-up (Glue spread, clamp, cure)
Billet Surfacing
Fabricate (Bore, dap, drill, trim)
Finish (Fill, seal, wrap, crate, mark)
Loading
Outbound Shipment

The laminations next move to the prepatching department where edge defects are removed in one stroke of a cutting tool and an interlocking patch, matching the grain and color of the wood, is inserted in the return stroke. This department eliminates much of the expensive finishing of a completed member that is required to meet the high quality appearance demanded of glued members for many uses.

The laminations then move to the scarfing department where optimum sloped scarfs are cut by planer knives cutting parallel to the grain to assure an end joint surface as smooth as obtainable, with absence of raised and torn grain which results from knives cutting across the grain. The scarfed laminations then move to the pre-gluing department where automatic positioning high-frequency machines bond the end joints in a matter of seconds with a continuous ribbon of laminations being produced.

The laminations are cut to the required length of the member and a "dry lay-up" made with proper segregation of grades and dispersal of end joints. The full length laminations with pre-glued end joints then move through a cabinet surfacer which planes the laminations to the final thickness and assures a uniformly smooth surface throughout the length and width with the required cleanliness of surface prior to gluing. The laminations then move to the gluing department and pass through the glue spreaders and into the forms where uniform pressure is applied and the member is shaped to the required camber or curvature.

Following the curing period, the clamps are removed and the member moves through pre-framing, and then through a giant surfacer capable of planing both surfaces of the $12\frac{1}{2}$-in. by $71\frac{1}{2}$-in. by 103-ft member previously mentioned, in a matter of 10 min. Final fabrication, patching and finishing, and application of surface sealers, wrapping, and identification complete the production cycle, and the member is ready for shipping.

Throughout the production schedule, the quality control department checks moisture contents, lumber gradation, surfacing, glue mix, spread and pressures, and other phases of the manufacturing process in accordance with the AITC "Inspection Manual," (3) to assure a quality product to meet the high industry standards. Included in the quality control check list are glue block shear tests conducted in accordance with ASTM Method D 805,[2] and cyclic delamination tests to determine the integrity of glue joints in laminated wood products for exterior service, conducted in accordance with ASTM Method D 1101.[3] The quality control department is responsible to management and is not under the jurisdiction of production. Quality control establishes the quality of the product, and the production department determines the procedure required to produce that quality.

CONCLUSION

A considerable expenditure for modern equipment to meet acceptable standards of quality and an abundance of "know-how" are obvious requisites of an up-to-date laminating plant. The ultimate in achieving the highest quality at a minimum cost has by no means been achieved, but constant research and development of new equipment and the need for keeping up with latest developments has all but eliminated marginal operations and sub-standard quality of products. The laminating industry stands ready to meet any new challenge calling for increased production or new products. It might well be said that the laminating industry has reached maturity, and while future trends and developments may result in such applications as paper or plastic overlays to new sections and structural shapes, the technical knowledge and production facilities available within the industry is adequate assurance that the position of timber as one of our major structural building materials will be continuously maintained and increased.

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[2] Methods of Testing Veneer, Plywood, and other Glued Veneer Constructions (D 805 – 52), 1955 Book of ASTM Standards, Part 4, p. 879.

[3] Method of Test for Integrity of Glue Joints in Laminated Wood Products for Exterior Service (D 1101 – 53), 1955 Book of ASTM Standards, Part 4, p. 954.

REFERENCES

(1) "Fabrication and Design of Glued Laminated Wood Structural Members," *Technical Bulletin No. 1069*, U. S. Forest Products Lab.

(2) "Timber Construction Standards," Am. Inst. Timber Construction.

(3) "Inspection Manual for Structural Glued Laminated Lumber," Am. Inst. Timber Construction.

(4) "Standard Specifications for the Design and Fabrication of Structural Glued Laminated Southern Pine," Southern Pine Assn.

(5) "Standard Specifications for the Design and Fabrication of Structural Glued Laminated Lumber," West Coast Lumbermen's Assn.

FACTORS AFFECTING STRENGTH AND DESIGN PRINCIPLES OF GLUED LAMINATED CONSTRUCTION

By Alan D. Freas[1]

Man's use of wood for structural purposes goes back untold centuries. From the fallen-tree bridge, the rude hut, and the hollow-log canoe, we have traveled a long road to the timber structures of the present day. With this progress has come also a change in conditions. When timber supplies seemed limitless, there was little reason for concern about the species, size, or quality of the timber to be used. Consequently, much timber construction was characterized by oversize members, frequently of very high grade.

As timber supplies became more limited and prices higher, precision of use became more important. The development of structural grades, while not offering a complete solution, was an important step in the more precise, economical, and efficient use of timber. The advent of glued laminated lumber improved still further the prospects for efficient utilization, since this lumber can be made from shorter lengths, lower grades, smaller sizes, and combinations of high and low grades.

Full realization of these prospects, however, depends more than ever on the greatest possible knowledge of the factors affecting strength so that sound principles of design may be developed. Since laminated lumber generally costs more than solid sawn timber, it must be used wisely and economically. To supply the basic information that would lead to the acceptance of laminated lumber, the U. S. Forest Products Laboratory began its research in this field more than 20 yr ago. The first major product of this research was the bulletin "The Glued Laminated Wooden Arch."[2] Although this bulletin was first published in 1939, it is still an authoritative publication on the subject. War-time experience demonstrated some deficiencies of knowledge and this led to an extensive program of research aimed at more precise knowledge of factors affecting strength. The research on laminated lumber was reviewed at the first ASTM Pacific Area National Meeting in 1949.[3] Later, the available data were analysed in the bulletin "Fabrication and Design of Glued Laminated Wood Structural Members."[4] Since this bulletin combines fabrication and design data, it is valuable to the designer, fabricator, and user alike.

It is the purpose of this paper to review the data and to describe the design principles derived from them. In view of the detailed presentation in 1949,[3] factors affecting strength will be covered in only a general way. The emphasis will be on design principles.

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[1] Engineer, U. S. Forest Products Laboratory, Madison, Wis.

[2] T. R. C. Wilson, "The Glued Laminated Wooden Arch," *Technical Bulletin No. 691*, U. S. Dept. of Agriculture (1939).

[3] Alan D. Freas, "Studies of the Strength of Glued Laminated Wood Construction," ASTM Bulletin, No. 170, Dec., 1950, p. 48 (TP 274).

[4] Alan D. Freas and M. L. Selbo, "Fabrication and Design of Glued Laminated Wood Structural Members," *Technical Bulletin No. 1069*, U. S. Dept. of Agriculture (1954).

Factors Affecting Strength

The same characteristics, such as knots and cross grain, that affect the strength of solid sawn timbers also affect the strength of laminated timbers. There are, however, additional factors peculiar to laminated construction that must be considered.

Glue Joint Quality:

Obviously, the bond between the laminations is of primary importance. Without that bond, there would be no structure, but only a group of laminations capable of serving only a limited structural purpose. For this reason, an adhesive must be chosen that will furnish such a bond, not only initially but over the period of service of the structure.

Today, a wide range of adhesives is available to satisfy very nearly any service condition from mild to extreme. Perhaps the most important of the newer adhesives from the standpoint of the laminating industry are the resorcinol resins and the mixtures or blends of phenol and resorcinol resins. On low-density species, resorcinol-resin adhesives may be cured adequately for many purposes at room temperatures. Phenol-resorcinol blends and resorcinol resins on high-density species require curing at elevated temperatures, if the laminated item is to withstand severe service. If properly handled, both types of adhesives are capable of withstanding adverse service conditions.

In many places, like the inside of covered buildings, service conditions are mild, and a highly resistant adhesive is not required. Under such conditions, casein adhesives are commonly used and may be expected to give long periods of satisfactory service.

Proper selection of an adhesive, however, is only one step in insuring a satisfactory glue bond. Proper techniques of use, which involve a variety of factors, are required. Among the more important factors are: uniform seasoning of the lumber, smooth and uniform surfacing of the laminations, proper mixing and spreading of the glue, adequate and uniform application of pressure, and proper curing of the glue. Proper curing includes proper temperatures and pressures and control of relative humidity during curing. The variety and character of these factors generally necessitate special plant equipment and special skills for the necessary control. Ordinarily, this will preclude on-site fabrication, particularly of important structural members.

Lamination Thickness:

Tests of a number of defect-free beams made with laminations of varying thickness indicated that lamination thickness had no effect on properties.

Knots:

A strength-reducing feature, such as a knot, will affect strength less if it is located in a region of low stress than if it is located in a region of high stress. Thus, the effect of a knot on the strength of a laminated beam is dependent on both its size and its position. This effect is best measured by moment of inertia. Results of a considerable number of tests have provided an empirical relationship between bending properties and the ratio I_K/I_G, where I_K is the moment of inertia of the areas occupied by all knots within 6 in. of the critical cross-section and I_G is the gross moment of inertia of the beam. A design curve derived from these data is shown in Fig. 1. The effect on modulus of elasticity (Fig. 2) is considerably less than the effect on bending strength.

The relationship between bending strength and the factor I_K/I_G suggests that it should be possible to use laminations with large knots in the central part of the depth of a beam between outer laminations with smaller knots without a serious loss of strength when compared with a beam in which all laminations contain the smaller knots. Tests have confirmed this view.

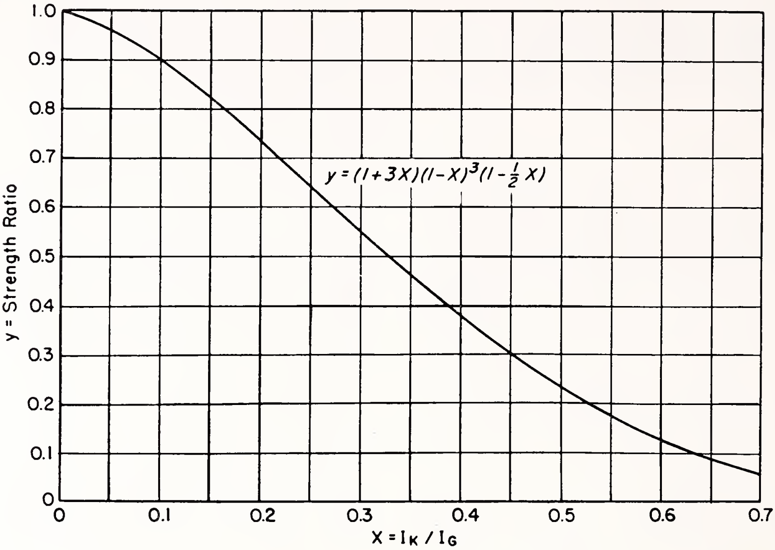


Fig. 1.—Design Curve Relating Allowable Flexural Stress to Moment of Inertia of Areas Occupied by Knots in the Laminations of Laminated Beams.

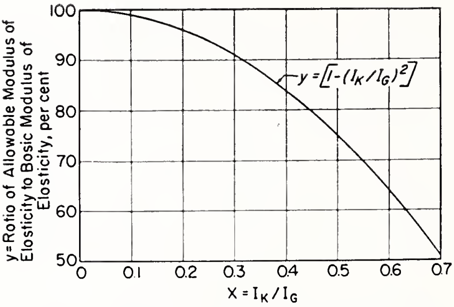


Fig. 2.—Design Curve for Laminated Beams with Knots in the Laminations Relating Allowable Modulus of Elasticity to Moment of Inertia of Areas Occupied by Knots.

In a column, there is no effect of position, since stress is uniform over the cross-section. Analysis of data from tests of columns indicated that the properties are proportional to the ratio of the average size of the maximum knot in each lamination to the width of the lamination (Fig. 3). A similar relationship would be expected to hold for tension members, except that the strength reduction for a given value of K/b would be somewhat greater.

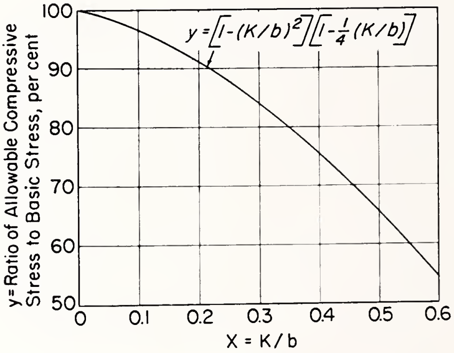
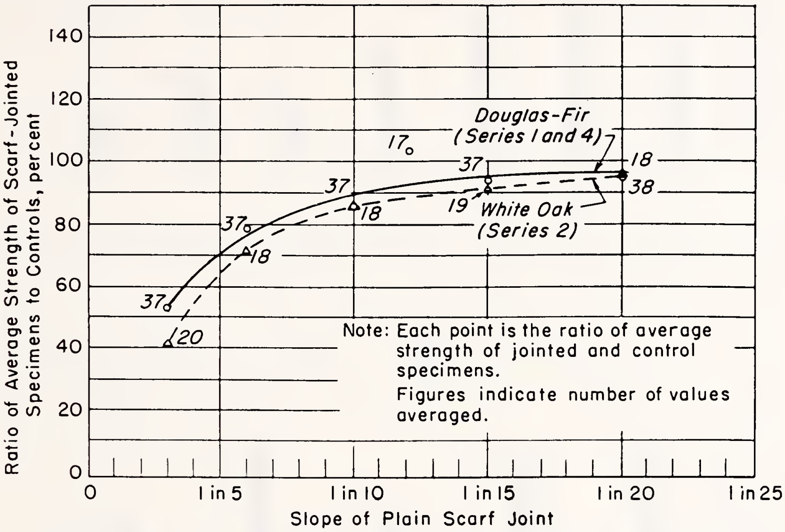


Fig. 3.—Design Curve for Laminated Columns with Knots in the Laminations Relating Allowable Compressive Stress to Knot Size.

Cross Grain:

There has been no systematic investigation of cross grain similar to that described for knots. It is obvious, however, that steeper cross grain could be permitted in areas of low stress than in areas of high stress in a beam. In the areas of high stress, the effect of cross grain would be expected to be similar to that in solid sawn timbers.



Note: Each point is the ratio of average strength of jointed and control specimens.
Figures indicate number of values averaged.

FIG. 4.—Maximum Strength in Tension Parallel to Grain as Related to the Slope of a Plain Scarf Joint in Douglas-fir and White Oak.

End Joints:

In laminated members of considerable size, pieces of lumber must be joined end to end to provide laminations of sufficient length. These joints are an important factor in determining the strength of laminated members.

Since stress cannot be transferred across a butt joint, such a joint represents an ineffective area, and additional cross-section must be provided to compensate for it. Tests of beams have shown that for butt joints in the compression side, the strength was approximated by computing the section modulus, as if the butt-jointed lamination were not present. When the butt joints were in the tension side of the beam, particularly in the outer lamination, an even greater effect was found. Strain measurements in the vicinity of a butt joint showed a severe stress concentration in adjacent laminations. Butt joints are not commonly used in important structural members. If they are, however, both the ineffective area they represent and their stress-concentrating effects must be taken into account.

Scarf joints, on the other hand, are effective means for joining the ends of pieces to form laminations of the required length. Even so, research has shown that scarf joints are not fully effective in tension. With their sloping surfaces, scarf joints are intermediate between the sidegrain glued joint, which can generally be made as strong as the wood, and the end-grain glued joint, which is characteristically weak and variable. Therefore, scarf joints are a compromise means of making an end-to-end joint with a minimum of end-grain gluing. Figure 4 illustrates the effect of scarf slope on joint strength. The steeper the slope, the greater is the proportion of end grain, and the lower the strength. In compression, scarf joint efficiency is much higher than in tension. Tests of columns containing scarf joints indicate that efficiency is on the order of 100 per cent even for slopes as steep as about one in five.

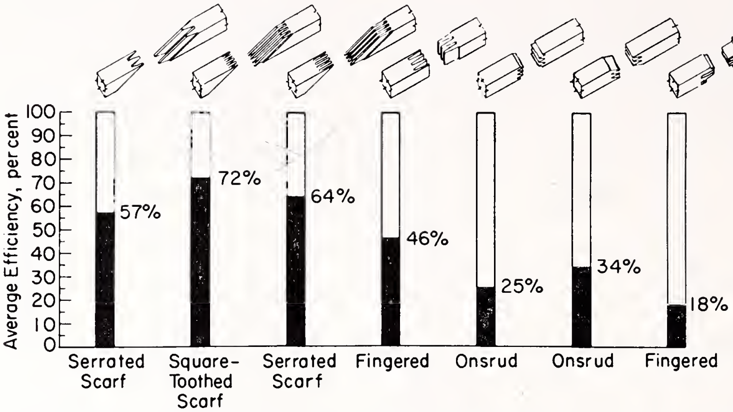


Fig. 5.—Comparison of the Strength in Tension Parallel to the Grain of Douglas-fir Specimens with Various End Joints and Corresponding Control Specimens.

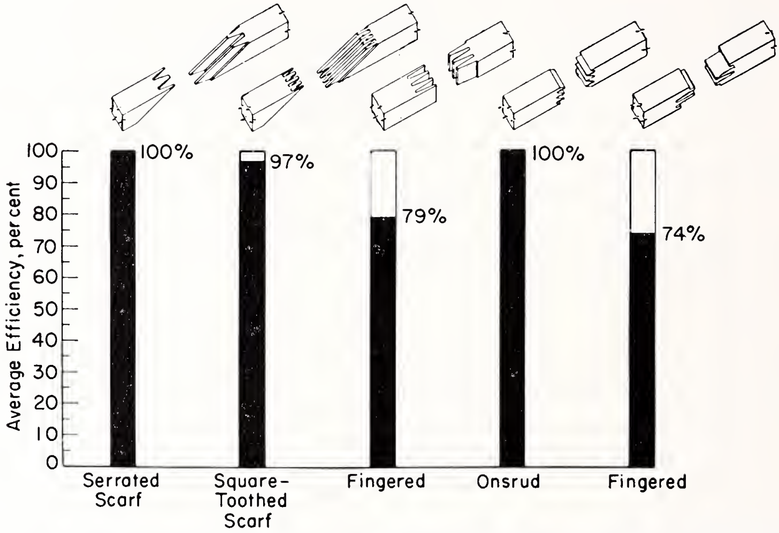


Fig. 6.—Comparison of the Strength in Compression Parallel to the Grain of Douglas-fir Specimens with Various End Joints and Corresponding Control Specimens.

Many varieties of end joints involving fingers of various types have been proposed and used, mainly in nonstructural applications. Their efficiencies vary considerably depending upon the form and slope of the fingers, but they are generally somewhat low. Figure 5 illustrates a few typical joints and their efficiencies in tension. Efficiencies under compressive stress are much higher (Fig. 6).

Stresses Induced by Bending Laminations to Curved Form:

In fabricating curved members, such as arches, curved beams, ship frames, and the like, stresses are induced in the individual laminations as they are bent to the curved form. The amount of stress will obviously depend upon the ratio of the radius of bend to the lamination thickness. Tests have indicated that the ratio of the strength of a curved member to that of a comparable straight member is given by the factor

$$1 - 2000\,(t/R)^2$$

where t is the lamination thickness, and R is the radius to which the lamination is bent.

The strength reduction indicated by this formula is considerably less than would be expected from a consideration of the magnitude of stresses induced by bending the laminations. For example, bending to a radius 160 times the thickness produces stresses on the order of one-half the ultimate and thus about equal to the proportional limit. Stresses at this level in individual laminations would be expected to cause severe strength reductions in the laminated member. Since severe reductions were not found in testing, it is apparent that the stresses induced in bending the laminations to form are relieved to a considerable extent. The data indicated that modulus of elasticity is not affected.

Height and Form of Bending Members:

It has long been known that stresses in wood beams, as computed by conventional methods, are affected by both the height and form of the cross-section. This has led to the development of empirical form and height factors to be applied to the usual bending formulas. The effect of height has heretofore been relatively unimportant, since the height that could be realized in solid timbers has been limited. In laminated construction, however, this limitation has been removed, and beams and arches of considerable height are common. Consideration of this effect thus assumes some importance in laminated structures.

DESIGN PRINCIPLES

Working Stresses:

In timber design, it has been the practice to assign basic stresses to the various species and to compute working stresses for particular grades by multiplying the basic stresses by a factor called the strength ratio. The strength ratio represents the proportion of the strength of a defect-free piece remaining after taking into account the effect of strength-reducing features, such as knots and cross grain. The same principle is used for developing working stresses for laminated structural members.

Basic Stresses.—The basic stress represents, essentially, the working stress applicable to a defect-free piece. It is derived from the average properties of the species by applying factors that adjust laboratory test results to actual conditions of use.

The same reduction factors that apply to solid sawn timbers will apply also to laminated timbers. Therefore, the basic stresses for laminated timbers used under service conditions involving high moisture content are the same as those for solid timbers. That is, they are based on strength of wood in the green condition (Table I).

One of the advantages of a laminated member is that it can be made of laminations small enough in cross-section to be seasoned readily before assembly. These laminations can then be assembled to form a member seasoned throughout and free from the tendency to check and distort after erection. Obviously, such a member also may be sufficiently dry throughout to justify the use of

TABLE I.—BASIC STRESSES FOR STRUCTURAL MEMBERS, LAMINATED FROM CLEAR MATERIAL AND UNDER LONG-TIME SERVICE AT MAXIMUM DESIGN LOAD AND UNDER WET CONDITIONS, FOR USE IN DETERMINING WORKING STRESSES ACCORDING TO GRADE OF LAMINATIONS AND OTHER APPLICABLE FACTORS.

Species[a]	Extreme Fiber in Bending or Tension Parallel to Grain, psi	Maximum Horizontal Shear, psi	Compression Perpendicular to Grain, psi	Compression Parallel to Grain, $L/d = 11$ or less, psi	Modulus of Elasticity in Bending 1000 psi
SOFTWOODS					
Baldcypress (southern cypress)	1900	150	220	1450	1200
Cedars					
Redcedar, western	1300	120	145	950	1000
White-cedar, Atlantic (southern white cedar) and northern	1100	100	130	750	800
White-cedar, Port Orford	1600	130	185	1200	1500
Yellow-cedar, Alaska (Alaska cedar)	1600	130	185	1050	1200
Douglas-fir, coast type	2200	130	235	1450	1600
Douglas-fir, coast type, close-grained	2350	130	250	1550	1600
Douglas-fir, Rocky Mountain type	1600	120	265	1050	1200
Douglas-fir, all regions, dense	2550	130	275	1700	1600
Fir, California red, grand, noble, and white	1600	100	220	950	1100
Fir, balsam	1300	100	110	950	1000
Hemlock, eastern	1600	100	220	950	1100
Hemlock, western (West Coast hemlock)	1900	110	220	1200	1400
Larch, western	2200	130	235	1450	1500
Pine, eastern white (northern white), ponderosa, sugar, and western white (Idaho white)	1300	120	185	1000	1000
Pine, jack	1600	120	160	1050	1100
Pine, lodgepole	1300	90	160	950	1000
Pine, red (Norway pine)	1600	120	160	1050	1200
Pine, southern yellow	2200	160	235	1450	1600
Pine, southern yellow, dense	2550	160	275	1700	1600
Redwood	1750	100	185	1350	1200
Redwood, close-grained	1900	100	195	1450	1200
Spruce, Engelmann	1100	100	130	800	800
Spruce, red, white, and Sitka	1600	120	185	1050	1200
Tamarack	1750	140	220	1350	1300
HARDWOODS					
Ash, black	1450	130	220	850	1100
Ash, commercial white	2050	185	365	1450	1500
Beech, American	2200	185	365	1600	1600
Birch, sweet and yellow	2200	185	365	1600	1600
Cottonwood, eastern	1100	90	110	800	1000
Elm, American and slippery (white or soft elm)	1600	150	185	1050	1200
Elm, rock	2200	185	365	1600	1300
Hickory, true and pecan	2800	205	440	2000	1800
Maple, black and sugar (hard maple)	2200	185	365	1600	1600
Oak, commercial red and white	2050	185	365	1350	1500
Sweetgum (red or sap gum)	1600	150	220	1050	1200
Tupelo, black (blackgum)	1600	150	220	1050	1200
Tupelo, water	1600	150	220	1050	1200
Yellow-poplar	1300	120	160	950	1100

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[a] Species names from approved check list, U. S. Forest Service, 1944. Commercial designations are shown in parentheses.

stresses based on the higher strength of dry material. This is true, however, only if the conditions of service are such as to maintain a low moisture content throughout the service life of the member.

Accordingly, a second set of basic stresses (Table II) is recommended for structures used under dry conditions. These stresses are derived by multiplying the basic stresses applicable to wet or moist conditions of service by a factor representing the effect of drying on the particular property. Since the effect of drying is not the same for all species nor for all pieces within a species, the factors were taken as approximately one-half the average increase in strength from the green to a 12-per cent moisture content condition as found from tests of small clear specimens. In some instances, the increases are substantial, as comparison of Tables I and II indicates.

Strength Ratio.—As was indicated earlier, there are available relationships between strength and knot size and, in the case of bending members, knot position. It is impractical to preassemble a member, so that knot sizes and positions are known in order to assign a design stress. It is equally impractical to assemble members with knots of specified sizes in specified positions. Rather, laminated members will be formed from random assemblies of laminations of a specific grade or combination of grades.

It is necessary, therefore, to find some means of assigning stresses to a random assembly and this suggests applying the principles of probability. From a survey of a typical sample of a grade, the frequency of occurrence of various knot sizes may be determined. By applying this information with the principles of probability and, for bending members, with consideration of the effects of knot position with respect to the neutral axis, the value of I_K/I_G or K/b, which has any given probability of occurrence, may be predicted. Choosing a level of probability considered suitable for design, these factors may be used to obtain strength ratios from the empirical relationships described earlier.

Since knots would be expected to be more dispersed in assemblies of large numbers of laminations than in assemblies of small numbers, an increase in strength ratio with increasing numbers of laminations would be expected. This

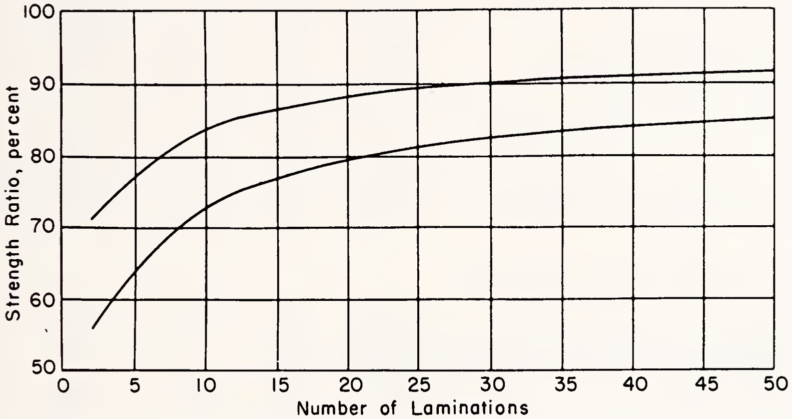

Fig. 7.—Sample Plot of Strength Ratios for Laminated Bending Members Containing Various Numbers of Laminations as Based on a Survey of Knot Size and Location. Each Curve Represents a Typical Lumber Grade.

TABLE II.—BASIC STRESSES FOR STRUCTURAL MEMBERS LAMINATED FROM CLEAR MATERIAL AND UNDER LONG-TIME SERVICE AT MAXIMUM DESIGN LOAD AND UNDER DRY CONDITIONS, AS IN MOST COVERED STRUCTURES, FOR USE IN DETERMINING WORKING STRESSES ACCORDING TO GRADE OF LAMINATIONS AND OTHER APPLICABLE FACTORS.

Species[a]	Extreme Fiber in Bending or Tension Parallel to Grain, psi	Maximum Longitudinal Shear, psi	Compression Perpendicular to Grain, psi	Compression Parallel to Grain, psi	Modulus of Elasticity in Bending, 1000 psi
SOFTWOODS					
Baldcypress (southern cypress)	2400	170	330	2000	1300
Cedars					
Redcedar, western	1600	135	220	1300	1100
White-cedar, Atlantic (southern white cedar) and northern	1400	115	195	1050	900
White-cedar, Port Orford	2000	150	275	1650	1600
Yellow-cedar, Alaska (Alaska cedar)	2000	150	275	1450	1300
Douglas-fir, coast type	2750	150	350	2000	1800
Douglas-fir, coast type, close-grained	2950	150	375	2150	1800
Douglas-fir, Rocky Mountain type	2000	135	310	1450	1300
Douglas-fir, all regions, dense	3200	150	410	2350	1800
Fir, California red, grand, noble and white	2000	115	330	1300	1200
Fir, balsam	1600	115	165	1300	1100
Hemlock, eastern	2000	115	330	1300	1200
Hemlock, western (West Coast hemlock)	2400	125	330	1650	1500
Larch, western	2750	150	350	2000	1600
Pine, eastern white (northern white), ponderosa, sugar, and western white (Idaho white)	1600	135	275	1400	1100
Pine, jack	2000	135	240	1450	1200
Pine, lodgepole	1600	100	240	1300	1100
Pine, red (Norway pine)	2000	135	240	1450	1300
Pine, southern yellow	2750	180	350	2000	1800
Pine, southern yellow, dense	3200	180	410	2350	1800
Redwood	2200	115	275	1850	1300
Redwood, close-grained	2400	115	295	2000	1300
Spruce, Engelmann	1400	115	195	1100	900
Spruce, red, white, and Sitka	2000	135	275	1450	1300
Tamarack	2200	160	330	1850	1400
HARDWOODS					
Ash, black	1800	150	330	1150	1200
Ash, commercial white	2550	210	550	2000	1600
Beech, American	2750	210	550	2200	1800
Birch, sweet and yellow	2750	210	550	2200	1800
Cottonwood, eastern	1400	100	165	1100	1100
Elm, American and slippery (white or soft elm)	2000	170	275	1450	1300
Elm, rock	2750	210	550	2200	1400
Hickory, true and pecan	3500	235	660	2750	2000
Maple, black and sugar (hard maple)	2750	210	550	2200	1800
Oak, commercial red and white	2550	210	550	1850	1600
Sweetgum (red or sap gum)	2000	170	330	1450	1300
Tupelo, black (blackgum)	2000	170	330	1450	1300
Tupelo, water	2000	170	330	1450	1300
Yellow-poplar	1600	135	240	1300	1200

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[a] Species names from approved check list, U. S. Forest Service, 1944. Commercial designations are shown in parentheses.

is illustrated in Fig. 7, which represents strength ratios computed from a probability analysis for a typical lumber grade. Combining defect-free or higher-grade outer laminations increases the strength ratio over that of a beam made wholly of the lower grade (Figs. 8 and 9).

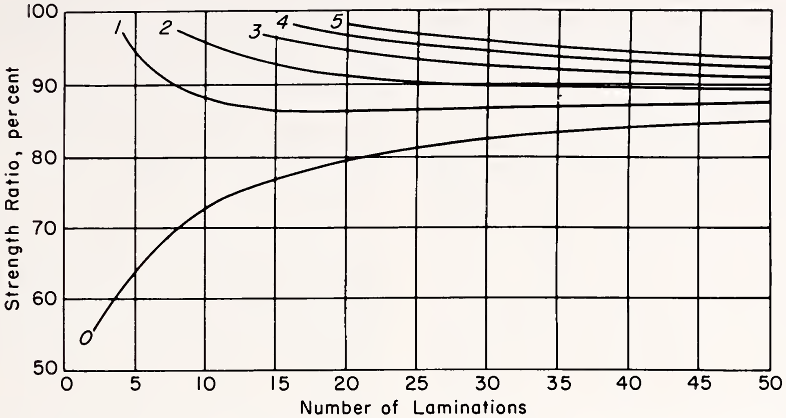

Fig. 8.—Sample Plot of Strength Ratios for Laminated Bending Members Containing Various Numbers of Laminations as Based on a Survey of Knot Size and Location.

A typical grade was used for the inner laminations and was combined with defect-free outer laminations. The curve numbers indicate the number of clear laminations on each side.

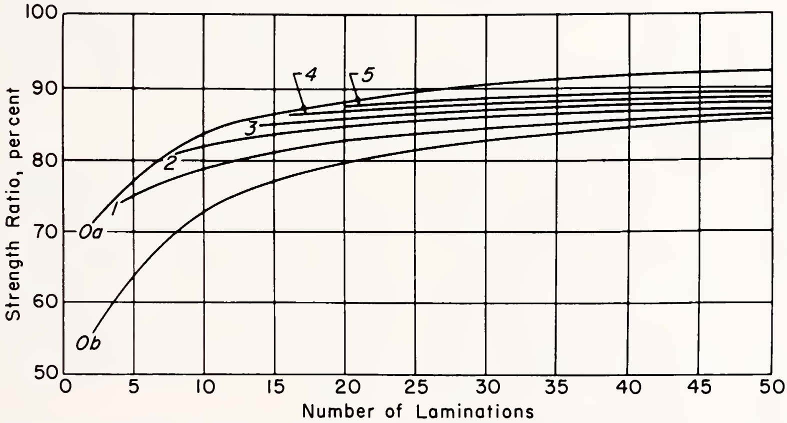

Fig. 9—Sample Plot of Strength Ratios for Laminated Bending Members Containing Various Numbers of Laminations Based on a Survey of Knot Size and Location.

Two typical grades of lumber were used in various combinations, and the curve numbers indicate the number of laminations on each side that are of higher grades, except curve *Oa* representing beams in which all laminations are of higher grade, and *Ob* in which all are of lower grade.

In establishing structural grades of solid sawn lumber, the limitations on knots and cross grain are set so as to give about the same strength ratio for each. Since knot dispersion results in a higher strength ratio, more restrictive limitations on cross grain must be imposed, at least in the outer groups of laminations. In this way, the higher strength ratio determined from the effect of knots may be realized.

Similarly, to assure that the end joints do not control the strength, the efficiency of the end joint chosen should not be less than that established on the basis of the occurrence of knots.

Solid one-piece timbers of large cross-section characteristically develop deep checks, as they season in service. Such checking reduces shear strength, because the area available to resist shear is reduced. Since laminated beams go into service thoroughly seasoned, they may be expected to check but little. Further, since the laminations are generally flat grained, such checks as may develop generally will lie in a vertical plane and have little effect on shear strength. For these reasons, design stresses for shear are commonly taken equal to the basic stresses. That is, there is no reduction for grade.

Design stresses proposed by lumber manufacturers associations have been developed from the principles described.

Methods of Analysis:

In general, methods of structural analysis applicable to structures of other materials are applicable also to structures of laminated wood. If curved members are involved, however, two problems may arise which are not encountered in the design of solid timber structures.

When the bending moment applied to a curved member acts to straighten it, tensile stresses perpendicular to grain are generated. When the bending moment tends to shorten the radius of curvature, the stresses perpendicular to grain are compressive. These stresses are maximum at the neutral axis and may be computed approximately from the formula:

$$S_R = \frac{3}{2}\frac{M}{Rbh}$$

where:

S_R = the radial stress,
M = the bending moment,
R = the radius of curvature,
b = the width, and
h = height of the cross-section.

Compressive stresses perpendicular to grain should be limited to the value shown in Tables I and II. Tensile stresses should be limited to about $\frac{3}{8}$ of the allowable shear stress for hardwoods and about $\frac{1}{3}$ of the allowable shear stress for softwoods.

The great depth possible in curved laminated bending members creates another problem in analysis that is not ordinarily encountered in timber design. It will be recalled from the principles of mechanics that stresses computed by ordinary methods for flexure will be in error for deep, sharply curved members. The error increases with the ratio of depth to centerline radius. In the majority of cases, the error will be small. However, this factor should be considered in analysis, and a check should be made for special cases by methods of analysis for curved beams.

Fatigue:

Concern is sometimes expressed over the possibility of fatigue failure of glued joints. Tests[5] have shown that well-made glued joints have fatigue characteristics similar to those for solid wood. Resistance to fatigue, therefore, is just as great in a laminated as in a solid member.

SUMMARY

More factors affect the strength of laminated structural members than affect the strength of solid sawn timbers. In general, however, these factors have been evaluated or are capable of control, so that design is on a firm footing.

The higher design stresses possible

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[5] Wayne C. Lewis, "Fatigue of Wood and Glued Joints Used in Laminated Construction, *Proceedings*, Forest Products Research Soc., Vol. 5 (1951).

with laminated timbers offer substantial advantages over solid timbers. However, this is only one of the factors that are bringing laminated construction into a prominent place in the building of churches, gymnasiums, bridges, farm structures, and in many other fields. The almost unlimited size possibilities, the possibility of fabricating curved members, and the freedom from excessive checking and distortion all play a prominent role.

Another important factor is the possibility this type of construction offers for the structural utilization of small sizes and low grades. As pointed out earlier, low grades can be effectively utilized in the lower-stressed portions of beams and arches. There should be equal opportunity to combine the lighter, lower-strength species with high-strength species, and research to develop design principles for this type of construction is under way.

In summary, it may be said that laminated construction, in spite of its phenomenal growth during the last two decades, has not yet demonstrated all of its possibilities. Imagination in design, coupled with advances in fabrication procedures and research, can be expected to broaden the horizons of laminated construction utilization.

STRENGTH QUALITIES OF DIMENSION LUMBER

By Lyman W. Wood[1]

Stress grading of lumber and working stresses for its use in structural design are well known. Most building codes give working stresses for wood. In the development of stress grading, principal emphasis was given to the larger timbers. More recently, increasing attention has been paid to the engineering design of light-frame structures and the strength values in the 2- or 3-in. dimension lumber used for framing. In response to this trend, many of the dimension grades of lumber have been redefined to make them stress grades. At the same time, not all dimension lumber is in stress grades, and engineers have demanded more information on the strength values in such lumber. That demand has led to the studies reported here.

It is necessary to point out the difference between stress-graded and non-stress-graded lumber. In stress grades, the characteristics that affect the strength are fully defined, and the grade is designed for an assured minimum strength value. Safe working stresses are based on minimum strength values and depend upon each individual piece to carry its own load. A safe working stress cannot be assigned to ungraded lumber or to lumber in a grade in which strength-affecting characteristics are not fully defined. Some pieces will have high strength and some very low. However, this non-stress lumber is widely used for framing members, such as joists and rafters spaced 16 to 24 in. on centers. Under such conditions, failure of an individual member does not portend collapse of the structure or danger to human life. Such use permits a somewhat different concept of acceptable working stress, recognizing the strength in a group of pieces rather than in the weakest individual piece. For example, the common floor system placed over a series of joists distributes a large portion of a load applied over one joist to the two adjacent joists. Thus, an average of three strength values may be considered in place of the minimum individual value.

A similar concept has been worked out and widely used in the design of glued laminated wood structural members. All laminations work together to support the load, and working stresses reflect a distribution of knots in adjacent laminations, recognizing that their occurrence in the same cross-section is highly improbable.[2] Joists and rafters are somewhat analogous to glued laminations in that the load is distributed to more than one member, and strength values may be considered on the distributed rather than the individual basis.

The most direct way to find the range of strength values is to test the strength of each piece in a sample large enough to represent adequately the whole lot under consideration. However, this is too costly. Existing knowledge of the strength values in clear wood of each species[3] and of the factors that reduce the strength of structural lumber below that of clear wood[4] is employed in an alternative method. In this method, the strength-reducing factors are evaluated visually on each piece in a large and representative sample. This evaluation gives a frequency distribution of strength ratios (the ratio of the strength of the piece to the strength of the clear wood in that piece). To illustrate, if a knot in a piece reduces its strength by 25 per cent, its strength ratio is 75 per cent. The frequency distribution of clear wood strength values is already known from standard strength tests.[3, 5] When the distributed strength ratio is multiplied by the distributed clear wood strength, a product distribution of estimated strength values in the lot of lumber is obtained.

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[1] Engineer U. S. Forest Products Laboratory, Madison, Wis.

[2] A. D. Freas and M. L. Selbo, "Fabrication and Design of Glued Laminated Wood Structural Members," *Technical Bulletin No. 1069*, U. S. Dept. of Agriculture (1954).

PURPOSE AND SCOPE

The purpose of the studies reported here is to show the range of strength values in dimension lumber not graded for strength. Results can be used in two ways: (1) to help in arriving at allowable working stresses that recognize the concept of distribution of load on light framing members, and (2) to suggest simple rules for selection to exclude the weakest pieces and thus improve the range of strength values.

This paper describes procedures and reports progress in two studies of this kind now under way. Data from a survey of ungraded dimension lumber used in farm buildings in the Northeastern states have been collected with the cooperation of the U. S. Northeastern Forest Experiment Station and the Agricultural Engineering Research Branch, U. S. Agricultural Research Service, and analysis of the data is under way. Similar data from a survey of 2 by 4-in. Douglas fir dimension lumber in two standard grades have been collected with the cooperation of the Oregon Forest Products Laboratory and the West Coast Lumbermen's Assn., and analysis of these data is essentially complete.

SURVEY DATA

The survey of strength ratios involves visual inspection of a number of pieces in a grade or size group of lumber at each of several points of production. In each piece, the critical defect controlling the strength is designated, and its effect estimated as a strength ratio. The principal strength factors considered are knots, slope of grain, off size, shakes and checks, and decay. Most of the survey work has been related to bending strength, and the position as well as the size of the defect has been considered. Figure 1 shows a sheet used for collecting survey data. The number of pieces surveyed at any point is optional, but 20 to 50 have been used in these studies.

Assignment of strength ratios is much like the conventional stress grading, but some differences exist. A stress grade admits a range of defects, and differences among pieces within that range are ignored. In the assignment of strength ratios, however, each piece has its own individual strength ratio, which may differ from that in any other piece. Another difference is that no pieces are rejected or culled. If a piece is in the sample, some strength ratio is assigned to it, no matter how low. Lumber in small sizes also requires more careful inspection of local distortions of grain, such as may result from sawing crooked trees or from large knots not present in the piece. This is generally not a problem in the larger timbers that account for much stress grading.

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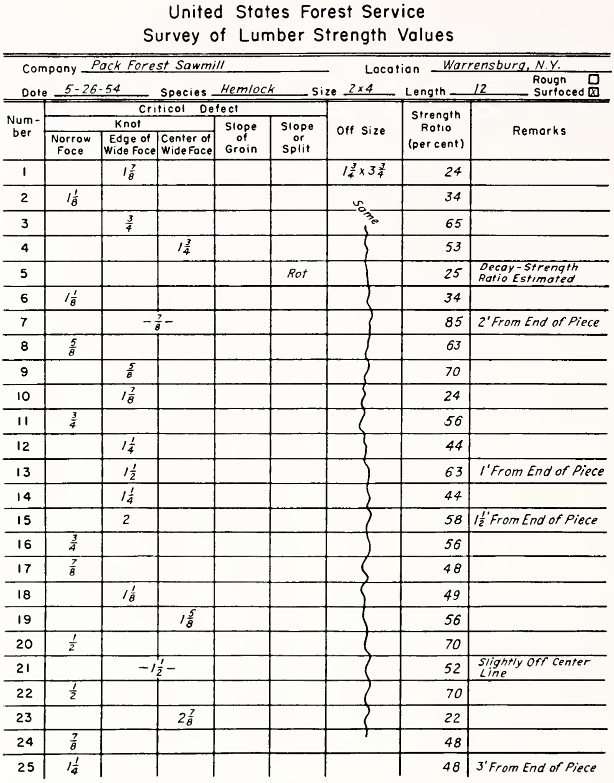
[3] L. J. Markwardt and T. R. C. Wilson, "Strength and Related Properties of Woods Grown in the United States," *Technical Bulletin No. 479* U. S. Dept. ofAgriculture, (1935).

[4] Methods for Establishing Structural Grades of Lumber (D 245 – 49 T), 1955 Book of ASTM Standards, Part 4, p. 997.

[5] Methods of Testing Small Clear Specimens of Timber (D 143 – 52), 1955 Book of ASTM Standards, Part 4, p. 823.

Laboratory strength tests are made on a limited number of the inspected pieces, the number ranging from 2 to 20 per cent of the total. The pieces are tested in full length, as inspected. After testing, small clear specimens are cut from lightly stressed portions of each piece to show the clear wood strength of that piece. The test strength divided by the clear wood strength gives the true strength ratio, which is compared with the estimated strength ratio from the survey. If the comparison should indicate some systematic error in the survey ratios, appropriate changes of survey practice or revisions of survey data are made.



United States Forest Service
Survey of Lumber Strength Values

Company *Pack Forest Sawmill* Location *Warrensburg, N.Y.*
Date *5-26-54* Species *Hemlock* Size *2x4* Length *12* Rough ☐ Surfaced ☒

Number	Critical Defect					Off Size	Strength Ratio (per cent)	Remarks
	Knot			Slope of Grain	Slope or Split			
	Narrow Face	Edge of Wide Face	Center of Wide Face					
1		$1\frac{7}{8}$				$1\frac{3}{4} \times 3\frac{3}{4}$	24	
2	$1\frac{1}{8}$					Same	34	
3		$\frac{3}{4}$					65	
4			$1\frac{3}{4}$				53	
5					Rot		25	Decay-Strength Ratio Estimated
6	$1\frac{1}{8}$						34	
7		$-\frac{7}{8}-$					85	2' From End of Piece
8	$\frac{5}{8}$						63	
9		$\frac{5}{8}$					70	
10		$1\frac{7}{8}$					24	
11	$\frac{3}{4}$						56	
12		$1\frac{1}{4}$					44	
13		$1\frac{1}{2}$					63	1' From End of Piece
14		$1\frac{1}{4}$					44	
15		2					58	$1\frac{1}{2}$' From End of Piece
16	$\frac{3}{4}$						56	
17	$\frac{7}{8}$						48	
18		$1\frac{1}{8}$					49	
19			$1\frac{5}{8}$				56	
20	$\frac{1}{2}$						70	
21		$-1\frac{1}{2}-$					52	Slightly Off Center Line
22	$\frac{1}{2}$						70	
23			$2\frac{7}{8}$				22	
24	$\frac{7}{8}$						48	
25	$1\frac{1}{4}$						48	3' From End of Piece

FIG. 1.—Sample Data Sheet Used in Field Surveys of the Strength of Dimension Lumber.

Figure 2 shows the distribution of differences (true ratios minus estimated bending strength ratios) in the pieces tested in these two surveys. In Fig. 2(*a*), nearly all of the differences in the Northeastern States survey were in the range of ±20 per cent. If the true strength ratio was 50 per cent, nearly all of the estimated strength ratios were in the range of 30 to 70 per cent. Figure 2(*b*) shows that only about $\frac{2}{3}$ of the differences in the Oregon survey were in the range of ±20 per cent. This was because the Oregon pieces were tested dry, and the estimated strength ratios included the expected effect of drying, which introduced an additional source of difference. In both studies, about as many pieces were underestimated as were over-

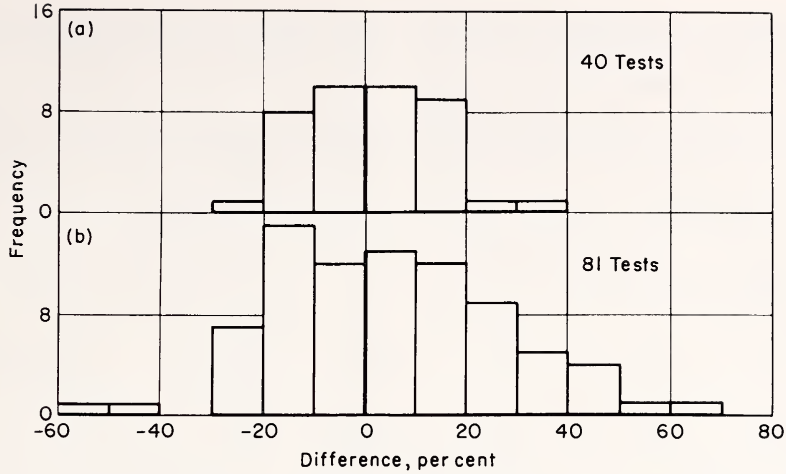


Fig. 2.—Distribution of Differences (True Ratios Minus Estimated Strength Ratios) in Dimension Lumber in Two Studies. (*a*) Northeastern States Study. (*b*) Oregon Study.

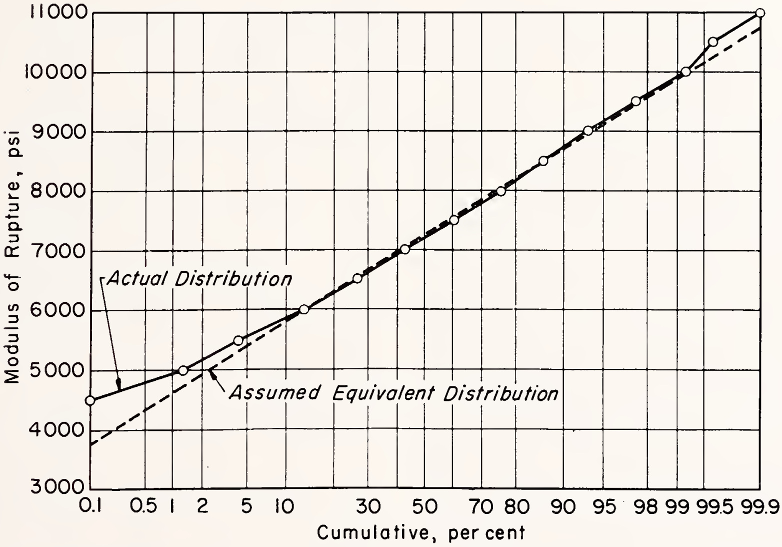


Fig. 3.—Actual and Assumed Equivalent Normal Frequency Distributions of Bending Strength of Green Clear Douglas-fir.

Number of tests—2038
Mean value—7247 psi
Standard deviation—1116 psi

estimated and by approximately equivalent amounts. This means that the estimated strength ratios, while considerably in error in some individual pieces, were rather close when considered as a distribution of a number of values.

Analysis of Data

The analysis of data may be summarized in three steps: (1) examination of the frequency distributions of clear wood strength and the distributions of strength ratios to show their character and indicate possible combinations of size or species groups, (2) calculation and examination of the product frequency distributions, and (3) application of possible rules for selection and examination of their effect on the product distributions.

Frequency distributions of clear wood strength values from standard tests are already known. Survey information in these studies indicates the distribution of strength ratios. These distributions are examined for normality and in the case of the strength ratios, for significance of the differences in mean values and standard deviations between the various size or species groups. The latter indicates which groups can be combined for further analysis.

All of the frequency distributions in the two studies reported here can be approximated by normal distributions without serious error. This approximation greatly simplifies the calculation of the product distributions. At the same time, there is some indication of a statistically significant departure from normality, which is being studied further. Figure 3 shows the actual frequency distribution of clear wood bending strength of Douglas fir on arithmetic probability paper. The chi-square test shows that this distribution departs significantly from normality. On the other hand, it can be seen that the straight line, representing the approximately equivalent normal distribution, is for the most part very close to the actual distribution.

Mathematical studies have indicated that an exact analytical solution for the distribution of the product of two uniformly distributed variates apparently does not exist. In the absence of an exact analytical solution, the product distribution can be defined by taking a number of random products of individual values of each variate. The multipliers going into these random products are chosen by suitable operations with a table of random normal numbers as indicated below. It has been found that 500 random products give satisfactory definition to the product distribution.

Random normal numbers represent various multiples of standard deviation in the frequency in which they occur in a normal distribution. The general range of such numbers is from −4 to +4. The numbers occurring most commonly are near zero, representing the most probable values in the normal distribution. Numbers exceeding 3, either plus or minus, are very rare. In use, the random normal numbers are divided into equal classes, such as +2.000 to +2.199, +2.200 to +2.399, and so on. For any normal frequency distribution, a multiplier is calculated to correspond to the midpoint of each class. That multiplier is then used in a random product in place of each random normal number drawn from that class. To illustrate, the assumed frequency distribution of clear wood strength of Douglas fir has a mean of 7247 and a standard deviation of 1116 psi (Fig. 3). The midpoint of the class of random normal numbers from −2.199 to −2.000 corresponds to $7247 - (2.0995 \times 1116) = 4903$ psi. This is used as a multiplier in place of all random numbers in the range of −2.199 to −2.000.

In like manner, the distribution of strength ratios in 12-ft Douglas fir 2 by 4's of No. 1 grade has a mean of 70.2 per cent and a standard deviation of 22.2 per cent. The midpoint of the class of random normal numbers from +1.200 to +1.399 corresponds to $70.2 + (1.2995 \times 22.2) = 99.0$ per cent. If a random normal number of −2.014 is drawn for clear wood strength and +1.209 is drawn for strength ratio, the pair gives a random product of $4903 \times 0.99 = 4854$ psi.

Application of a rule for selection of lumber has the effect of excluding some individual values and thus of modifying both the mean and the standard deviation of the frequency distribution of strength ratios. These in turn affect the product distribution. Mathematical means for evaluating that effect are now under study at the Oregon Forest Products Laboratory.

TABLE I.—CHARACTERISTICS OF FREQUENCY DISTRIBUTIONS OF BENDING STRENGTH RATIOS IN NORTHEASTERN STATES SURVEY OF DIMENSION LUMBER.

Group			Number of Values	Strength Ratio	
Species	Size, in.	Finish		Mean, per cent	Standard Deviation, per cent
Eastern hemlock	2 by 4	Dressed	350	53.0	16.0
	2 by 8	Rough	217	55.9	17.7
	2 by 8	Dressed	246	60.4	16.9
White pine	2 by 4	Rough	300	50.7	17.2
	2 by 4	Dressed	150	47.6	15.1
	2 by 8	Rough	200	56.0	18.2
	2 by 8	Dressed	210	55.8	19.6
Eastern spruce	2 by 4	Dressed	275	56.9	14.1
	2 by 8	Rough	214	61.9	14.6
	2 by 8	Dressed	250	53.6	19.6
Red oak................	2 by 4	Rough	200	62.5	17.7
	2 by 8	Rough	225	68.4	17.2

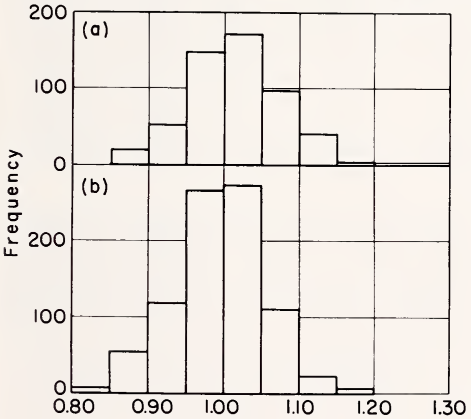


Fig. 4.—Frequency Distribution of the Ratio of Actual to Nominal Section Modulus in Rough Dimension Lumber in the Northeastern States Study. Ratio for (*a*) is for 2 by 4-in. size, ratio for (*b*) is for 2 by 8-in. size.

Northeastern States Survey

Much dimension lumber for farm-building construction in the Northeastern states goes directly from manufacturer to user without passing through the normal channels of retail trade or selection under standard grading procedures. Agricultural engineers concerned with farm-building design need information on the strength values in such lumber. The Northeastern States survey was planned to give that information.

Table I shows the mean values and the standard deviations of the distributions of strength ratios in the various survey groups from the Northeastern States study. These are groups of ungraded material in three softwood and one hardwood species, and differences among the groups are not large. Oak appears to be somewhat higher in average strength ratio than the softwoods. There is some indication that the strength ratios are higher in the 2 by 8-in. than in the 2 by 4-in. size.

Rough lumber shows a variation in size which can be an important strength factor. The effect of off size on bending strength can be expressed as a ratio of the actual to the nominal section modulus of the cross-section. Figure 4 shows frequency distributions of those ratios in the 2 by 4-in. and the 2 by 8-in sizes of rough dimension lumber in this study. Although a few pieces depart rather widely from nominal size, 85 to 90 per cent in both sizes have section moduli within 10 per cent of nominal.

Table II indicates the characteristics of all pieces in the Northeastern States survey with estimated strength ratios below 45 per cent. These were chosen somewhat arbitrarily to represent low-strength pieces because the lowest standard stress grades have that strength ratio. Table II shows that knots at the edges of wide faces predominate as a cause of low strength in the 2 by 4-in size while knots in narrow faces are more important in the 2 by 8-in. size. Slope of grain is shown to be relatively unimportant. Differences between 2 by 4's and 2 by 8's and between oak and the Northeastern softwoods may be noted in the last line of Table II.

TABLE II.—OCCURRENCE OF DEFECTS IN LOW-STRENGTH PIECES[a] IN SURVEYS OF DIMENSION LUMBER.

Defect Causing Low Strength	2 by 4-in. Pieces				2 by 8-in. Pieces	
	Northeastern Softwoods, per cent	Northeastern Oak, per cent	Douglas-Fir		Northeastern Softwoods, per cent	Northeastern Oak, per cent
			No. 1 Grade, per cent	No. 2 Grade, per cent		
Knot at edge of wide face	13.5	9.5	6.7	20.2	3.5	1.8
Knot away from edge of wide face	4.4	3.5	1.4	3.8	0.7	0
Knot in narrow face	10.9	4.5	1.0	5.3	16.9	5.8
Slope of grain	2.0	0	0	0.5	0.7	0.4
Other	0.9	0	0	1.9	0.3	0.4
Total	31.7	17.5	9.1	31.7	22.1	8.4

[a] Low-strength pieces were estimated to have bending strength ratios less than 45 per cent.

Product distributions to show the estimated strengths of this material are now being calculated.

OREGON SURVEY

While most 2 by 4-in. lumber is used in light framing where grading to a specified stress is not required, some is used in light trusses or otherwise where working stresses must be assigned. The Oregon survey of the 2 by 4-in. size was prompted by the need for strength values to which working stresses could be related. The grades were No. 1 and No. 2 (now Construction and Standard) Douglas fir. Two lengths, 8 and 12 ft, were surveyed. Ten pieces in each length and grade were surveyed at each of 10 mills, giving four size or grade groups of about 100 pieces each.

Since the majority of the Douglas fir 2 by 4's are used continuously dry as in most covered structures, the survey estimates were made to show the strength of dry material. That is, the observed strength ratios were increased to reflect the expected strength increase from drying. Effects of drying on structural lumber are variable and are hard to estimate accurately. The increases of strength ratios for drying thus introduced an additional element of uncertainty which is reflected in the greater differences shown in Fig. 2(*b*) compared to Fig. 2(*a*).

Characteristics of low-strength pieces in the Oregon survey are given in Table II. Knots at the edges of wide faces predominated, and slope of grain was relatively unimportant. Douglas fir in this respect was similar to the Northeastern

States softwoods. The last line of Table II shows how many more low-strength pieces were in the No. 2 than in the No. 1 grade.

TABLE III.—DISTRIBUTION OF ESTIMATED BENDING STRENGTH VALUES IN TWO GRADES OF DOUGLAS-FIR 2- BY 4-IN. DIMENSION LUMBER.

Estimated Modulus of Rupture, psi	No. 1 Grade			No. 2 Grade		
	Frequency	Cumulative Frequency	Cumulative Frequency, per cent	Frequency	Cumulative Frequency	Cumulative Frequency, per cent
0 to 490	1	1	0.2	2	2	0.4
500 to 990	0	1	0.2	9	11	2.2
1 000 to 1 490	6	7	1.4	17	28	5.6
1 500 to 1 990	8	15	3.0	32	60	12.0
2 000 to 2 490	18	33	6.6	43	103	20.6
2 500 to 2 990	23	56	11.2	41	144	28.8
3 000 to 3 490	32	88	17.6	65	209	41.8
3 500 to 3 990	48	136	27.2	50	259	51.8
4 000 to 4 490	62	198	39.6	63	322	64.4
4 500 to 4 990	54	252	50.4	49	371	74.2
5 000 to 5 490	57	309	61.8	41	412	82.4
5 500 to 5 990	51	360	72.0	34	446	89.2
6 000 to 6 490	37	397	79.4	20	466	93.2
6 500 to 6 990	31	428	85.6	13	479	95.8
7 000 to 7 490	27	455	91.0	10	489	97.8
7 500 to 7 990	18	473	94.6	4	493	98.6
8 000 to 8 490	11	484	96.8	4	497	99.4
8 500 to 8 990	7	491	98.2	2	499	99.8
9 000 to 9 490	1	492	98.4	0	499	99.8
9 500 to 9 990	1	493	98.6	1	500	100.0
10 000 to 10 490	2	495	99.0	...	...	...
10 500 to 10 990	1	496	99.2	...	...	...
11 000 to 11 490	1	497	99.4	...	...	...
11 500 to 11 990	1	498	99.6	...	...	...
12 000 to 12 490	0	498	99.6	...	...	...
12 500 to 12 990	2	500	100.0	...	...	...
Total	500	...	...	500	...	...

No. 1 Grade

Mean	5086 psi
Standard deviation	1823 psi

No. 2 Grade

Mean	3962 psi
Standard deviation	1664 psi

Mean values of survey strength ratios in the four groups showed a large and statistically significant difference between the two grades and a small, but statistically significant difference, between the two lengths. Standard deviations did not differ significantly. In the analysis, therefore, a common value of standard deviation (the average of the four groups) was used. Separate analyses were made on the two grades.

The 12-ft pieces in each grade were used for constructing two product distributions. In view of the small differences between 8- and 12-ft survey groups, it was felt that a working stress for the one length could be modified for the other through the relation of their mean values, thus obviating the necessity for running four product distributions. The 12-ft length was chosen, partly because of slightly lower strength ratios but also because stress uses like rafters or truss members are more likely to require that length. Mean values of strength ratios in the 12-ft length were 70.2 per cent in the No. 1 grade and 54.4 per cent in the No. 2 grade. The common value of standard deviation of strength ratios was 22.2 per cent. These values defined the two normal frequency distributions that were assumed to approximate the actual distributions of strength ratios.

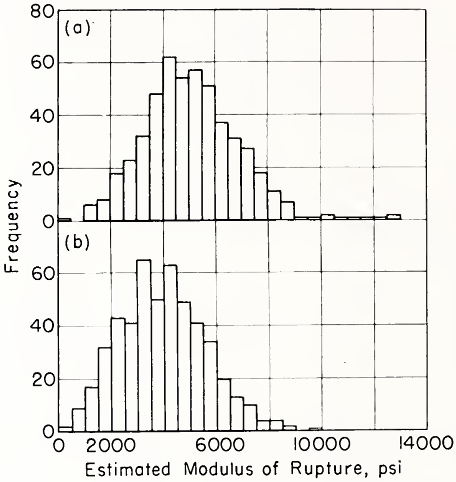


FIG. 5.—Distribution of Estimated Bending Strength Values in Two Grades of Douglas fir 2 by 4-in. Dimension Lumber. (a) No. 1 Grade, (b) No. 2 Grade.

Random products of these distributed strength ratios multiplied by the distributed clear wood strength values (Fig. 3) are classified in Table III and shown graphically in Fig. 5. These are the expected distributions of bending strengths in the No. 1 and No. 2 grades of 2 by 4's. Figure 5 shows the range of expected strength values in both grades and the overlap between the two. In both distributions, there is some skewness, with elongated tails at the upper ends. Statistical analysis of third and fourth moments of these distributions showed that both skewness and kurtosis (flattening and spreading of the peak) were significant in the No. 1 and that skewness was significant in the No. 2 grade. Trial plotting on probability paper indicates that these departures from normality are relatively small, though statistically significant. The possibility of approximating these product distributions by normal distributions is being considered.

CONCLUSION

Useful though these distributions of strength values are, they are not the final objective of studies of this kind. The distributions indicate a wide range of strength values, emphasizing the difficulty of assigning a working stress without stress grading. While joists and rafters spaced 16 to 24 in. on centers may permit consideration of the average of 3 instead of the weakest single piece, there is still a wide distribution of strength values based on such an average.

The survey data show the characteristics of low-strength pieces of dimension lumber. Further analysis should be made of the effect on the strength distribution of the application of a grading rule to exclude some or most of the low-strength pieces. Such a grading rule must be simple and easily applied if it is to be effective with dimension lumber. Studies of this possibility are under way.

Structural designers are increasingly aware of the value of statistical procedures in helping them to deal with the variability of structural materials. This paper indicates one such approach to an important problem in timber design. It is hoped that this report of the concepts and the techniques used will be helpful to structural engineers and that some contribution may be made thereby to the safe and economical use of structural lumber.

DISCUSSION

MR. T. K. MAY.[1]—The lumber industry is very much interested in the development of stress grading. Further work is needed to evaluate the information and to find a way of applying it to dimension grades of lumber. One of the biggest problems in grades of lumber for consumer acceptance is appearance. As is well known, the smaller sizes are used in conventional light framing. For such use the material passes through the hands of many non-technical people, right down to the realtor who sells the house and to the occupants of the house, none of whom will have any knowledge of stress values, nor are they cognizant of the factors that make for stress qualities.

This means that in the marketing of the smaller sizes the manufacturer has to emphasize appearance quality in lumber grades to the detriment of the stress qualities. If both appearance and stress qualities are a grade requirement, then there is the problem of increased cost, reflected to the consumer, because of the greater fall-down of adequately strong pieces into lower grades for appearance reasons, or a fall-down into lower grades for strength reasons.

We know, for example, that even technical people misunderstand those quality requirements that provide strength. We have had much experience in putting on programs for organizations where the audience participates in a lumber grading test. We always include among the test samples one piece that is clear but having a very steep slope of grain, so steep that it will not qualify for any stress grade. Almost invariably building officials, architects, engineers, and others, will prefer that piece to a very high-grade piece because it appeals to their eye. The same applies to a piece of lumber with many knots of comparatively small size; such a piece will be down-rated in comparison with a piece having one or two rather large knots, yet the small-knotted piece is stronger.

The lumbermen have another practical problem with the narrower widths. For example, the 2 by 4-in. size, when stress-graded, requires a smaller knot than wider widths for equal strength. This means that wider widths are more easily produced for the same stress quality and that the narrower width would be more costly to manufacture. However, the buyer objects to the price difference on comparable strength value, regardless of width. These are some of the reasons that most of the 2 by 4-in. size produced in the Douglas fir region is not stress-graded. The industry realizes the need for stress-graded 2 by 4's in some special uses, but these uses are limited and small volumes of special grades result in economic problems of inventory. As it is, there are too many lumber grades today for economical, dealer inventory.

These practical problems of appearance *versus* strength makes the statistical approach to a realistic strength value for non-stress grades of lumber of great interest.

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[1] Director of Technical Service, West Coast Lumbermen's Assn., Portland, Ore.

NEW DEVELOPMENTS IN ENGINEERED WOOD DESIGN AND CONSTRUCTION

By Verne Ketchum[1]

This paper applies only to engineered timber construction using sawn lumber and timbers as they come from the mill and planing mill. It does not directly cover glued laminated construction, although some items may also concern glued laminated work.

Sawn lumber and glued laminated members are often used in combination and adjacent to each other, such as sawn purlins and joists supported on glued laminated girders or arches. Many modern timber trusses include sawn webs and glued laminated chords. Small accessory pieces are often sawn lumber, though the main members are glued laminated pieces. In use, there can be no clean separation between sawn and glued laminated work.

In the modern usage of engineered timber construction there are many improvements such as treatments, timber connectors, improved accessories and developed combinations. Those items which are not of recent origin and are accordingly well known to all constructors will not be discussed.

In the broad fields of timber construction, there are perhaps four general classes of improvement of note. Timber construction has developed the new upsurge because of (1) expansion of timber fabrication business, (2) increase in availability of design and construction techniques, (3) use of more efficient fastenings, both mechanical and structural glued joints, and (4) more realistic recognition of the fire durability of timber.

The one-story type of building has been a major factor in the current popularity of timber construction. The one-time multi-story and basement masonry exterior wall and interior heavy timber frame are quite unknown today in timber construction.

In a broad appraisal of timber construction and its future developments, it would seem that we now have adequate technical knowledge. Grades, lumber qualities, and working strengths are well known and are based on good tests and careful analysis. Working stresses are now as high as the material warrants and no extensive or radical changes are being planned. There is, however, more factual test data needed on strengths of timber fastenings such as bolts, on which there is not yet final agreement.

There is great need for better recognition in codes since, because of old-time restrictions, timber framing is still prohibited in many regions for schools and other public structures. Height, area, and zone restrictions are often excessively prohibitive. In some states the use of timber construction is prohibited for school construction.

ADVANTAGE OF TIMBER CONSTRUCTION

Lumber and timbers can be planed, sawn and joined with simple carpenter tools. For this reason wood construction is the handiest and most easily worked construction material. For these same reasons, repairs, additions, and replacements are simple problems of construction. New or repair work in lumber requires little technical knowledge. Lumber and timbers are available everywhere from the neighborhood yard and can be quickly transported by truck or car trailer.

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[1] Chief Engineer, Timber Structures, Inc., Portland, Ore.

While lumber grading on large projects is usually performed by experienced graders, all users are generally qualified to determine the more important characteristics of lumber. The common shapes of shiplap sheathing, 2 by 4-in. studding and 2 by 10-in. joists, are all well known. Siding, shingles, moldings and flooring of wood are all familiar to us.

Timber construction using modern techniques can be strong and durable. Often timber construction for walls, roofs, and bridges is much lower in cost than those built of concrete or steel.

Wood framing and especially heavy timber construction will often withstand fires better than unprotected steel construction. For certain electrical and nonmagnetic uses wood can be used where steel cannot. In brief, there are many and varied uses where timber construction is superior and cheaper in cost than other building materials.

DISADVANTAGES OF TIMBER CONSTRUCTION

Timber construction has some disadvantages in that it is combustible, that, unless specially treated, it is subject to decay when exposed to certain conditions, and that it has a larger bulk for equal strength. Timber framing has less stiffness and more deflection under equal loads than does steel or concrete. Construction through modern fastenings and improved engineering has greatly improved these qualities in timber.

Timber framing does not have the wearing qualities against abrasion, nor is it generally suitable for watertight containers, nor can it be suitably embedded in concrete unless specially treated. Some minor embedment is done, however, and concrete bridge slabs often rest on timber framing.

While timber spans have been gradually lengthening, it may still be said that, in general, steel construction is usually more suitable for long span bridges and especially those of over 200 ft.

PREFABRICATION

The moving of timber framing operations from field to shop started several years ago, but recently this trend has been intensified and today nearly all heavy timber construction is shop built and, in combination with connecting hardware and iron work, is shipped, knocked down, to the job site. Often the fabricator includes assembly and erection at the job, thus combining and concentrating responsibility for the final complete performance. Perhaps the larger percentage of timber design today is performed by the fabricator, but subject to the approval of the architect or practicing engineer.

The fabricators have improved the product and lowered costs to increase production. Here, in a large measure, the use of automation has been responsible for expansion. At present all large fabricators produce much glued laminated work, and the gluing of structural work may often form the bulk of the fabricator's work. The principles of operation still apply to the sawn lumber fabricating departments of the business.

Large planers, automatic saws, and roll case in-line setups together with quality control have contributed largely to lowered costs, increased production, and improved quality. The industry is rapidly becoming standardized. Continuous inspection and the availability of company or institute certificates of conformance to job specifications are now common practice.

TIMBER LITERATURE

The 1955 edition of Wood Handbook which contains basic information on wood as a material of construction with data for its use in design and specification is a real addition to our timber construction literature. This handbook has been prepared by the U. S. Forest Products Laboratory.

The Timber Engineering Co. of Washington, D. C., has prepared a comprehensive Teco Timber Handbook which has been written by the timber experts of the United States under the editorship of Ralph Gloss.

The West Coast Lumbermen's Assn. is now putting the final touches on a new edition of its excellent Douglas Fir Use Book, which has been popular for many years with timber designers. There has been a scarcity over the years of design data and tables on arches, but the new publication will contain many pages of arch and lamella design. The new books, now out and in preparation, such as Douglas Fir Use Book and Teco Timber Handbook, will give the designer and constructor a wealth of ready references that a few years ago had to be laboriously accumulated by each designer.

The National Design Specification for Stress Grade Lumber and Its Fastenings has just been brought up to date with a new 1955 edition. New grading rule grades and stress values are shown. Previously published working stress values of sawn lumber are repeated in the late edition of NDS, except that the modulus of elasticity has been increased from 1,600,000 to 1,760,000 psi on the basis that construction lumber is at least partially seasoned before being subjected to full load.

A definite improvement in design that will be appreciated by all timber men is the simplified column formula. No longer need the computer fight through two-story column formulas to secure a refinement that is quickly lost to stock sizes of timber. The new column formula may be found in NDS and in Wood Columns, both by National Lumber Manufacturers Assn. of Washington, D. C.

NEW GRADING RULES

The West Coast Lumber Inspection Bureau of Portland, Ore. recently has issued a new Standard Grading and Dressing Rules No. 15 which supersedes previous Rules No. 14 for Douglas fir, West Coast hemlock, Sitka spruce and western red cedar. The new rules show names in some instances as replacements for the previous grade members. Lumber will be known now as Select Structural, Construction, Standard, Utility, and Economy. Also, there is now a division entitled "Light Construction" which applies principally to residence construction wherein experience and rules of thumb have long governed grades and sizes.

The working stresses of lumber in the new rules are similar to those previously used, although there are some minor changes which will be found in the shear tables. These rules apply principally to sawn lumber, although they can and are being adapted to laminations for glued construction.

Southern Pine Inspection Bureau has also issued new grading rules and has revised, somewhat, the working stresses therefor. For Dense Structural kiln-dried southern pine sawn lumber, there are listed working stresses in bending and tension parallel to grain values as high as 3000 psi. The descriptive name of Long Leaf has been deleted.

These high working stresses of southern pine do not normally apply to general timber construction. The medium high values of 2000 to 2400 psi are more nearly the ones to use for both Douglas fir and southern pine. High bending stresses are valueless where shear and deflection govern and high grades may be difficult to secure, and, if sights are raised too high, delay and confusion results.

HEAVY DECKING

In the past few years, there has been constructed and promoted a type of double tongue and groove decking which forms an excellent accessory to the modern glued laminated girders and arches. Of 3 in. and 4 in. in depth it easily spans 12 ft to 20 ft with standard roof loads and provides a smooth under-surface without the use of rafters, purlins or joists, and provides a high resistance to passage of sound, heat, cold or fire.

This double-tongued decking is generally available from fabricators under various trade names, with ends cut square and with spike holes predrilled at approximately 30-in. intervals through the wide dimension of the piece, all ready for laying and spiking in place as a continuous mat. In dimension, this decking is generally now in 3 by 6 in., and 4 by 6 in. nominal sizes, which rapidly are replacing the former 3-in. and 4-in. by 5-in. styles which did not fit normal sawing practices well. There is necessarily some variation in load-carrying capacity according to the pattern of the lengths as laid over the supports, and, to secure maximum results, the application of normal engineering practice is necessary. Corners on the face side of the decking are usually given a V-bevel cut to improve the appearance of the finished work. This material is dried to a moisture content of 12 to 15 per cent, as compared to the ordinary commercial dry range of 19 to 20 per cent, under which condition ordinary sheathing commonly goes to market.

Installation of this prepared decking is quite easily and quickly accomplished, and the variation and number of species from which the material may be manufactured gives the designer an attractive choice of woods and finishes. First he has the choice between richly figured western red cedar and the lighter colored woods which fall into what is termed the blond category. This decking can also be furnished from the better known structural species such as Douglas fir and southern pine and no doubt some other species. However, this can be done only at considerable additional cost and, in fact, one of the favorable features of the movement is that so-called secondary species, structurally speaking, are furnished with an additional market for which their material has superior values with respect to lesser weight and better insulating values than do the primary structural species. The natural result, then, is that the various species are better directed towards their respective economic qualifications.

The designer may also choose between smooth finish and striated surface and wire-brushed surface, though the latter is currently available only in cedar and hemlock. Striating, which consists of grooving the surface longitudinally with parallel channels about $\frac{1}{8}$ in. wide and $\frac{1}{2}$ in. apart, produces a soft and rather draped appearance when in place on walls or ceilings. It also improves acoustics and frequently is popular for churches and other places where echo might otherwise be a problem. The wire-brushed surface brings the grain into higher relief, something like sandblasting, and is useful for special effects.

Heavy decking is one of the real new improvements in sawn timber construction and many times it carries out a bulk and all-wood appearance that otherwise could not be obtained with arches and beams alone.

TIMBER TRUSSES

Sawn lumber has served well in the construction of a myriad of timber trusses, bowstring, parallel chord, A-frame, monochord and a host of other types. Various leading fabricators have developed their own individual types of trusses, all of which have given fine service, and the competition thus generated has without doubt lowered building costs and contributed to the economical provision of the wide-span, unencumbered floor space now so much in demand in American building.

As a late development there has been a withdrawal from the construction of sawn chords, particularly lower chords in tension, by at least one large fabricator. Glued laminated chords are used exclusively in this instance, but sawn members are still used for the webs or intermediate members such as joists, purlins, and bracing. The reasons for the change are no reflection on past performance and base the action upon the changes which have taken place in the sawmill picture, by reason of changing in logging practices and of diversion of high-grade logs to the plywood industry. This has resulted in high cost and slow delivery of the lengths and grades considered necessary for sawn chords, to the extent that it was found best to change to the glue laminating of these members, whereby any size, length, or grade at any time could be produced. Contributing factors included removal of sales resistance due to maintenance factors, such as bolt tightening, occasional member replacement due to excessive checking or splitting, and appearance factors.

Wood trusses, like other timber framing, have gone through some evolutionary improvements. General design is usually according to accepted standard practice that has been with us for some years. Types of members and details have, however, been changed and improved. Members and details have been better adapted to styles which avoid, as much as possible, shrinkage and maintenance. The use by some fabricators of strap and pin connections is a decided improvement in these directions. Bolted rather than timber connector joints are used extensively in some areas.

Seasoned lumber is not usually available commercially in sufficient quantities for the average project. This is the reason why much heavy timber construction uses unseasoned lumber which is permitted to season in place. Trusses installed unseasoned can have a satisfactory life, but should have a certain amount of inspection and servicing until the framing has reached moisture equilibrium.[2, 3]

Shortly there will be printed a new standard on Timber Trusses and Bracing by the American Institute of Timber Construction.

BUILT-UP AND BOXED COLUMNS

Built-up columns have been used for many years, and strength and design data are given in the Wood Handbook. The late edition of this publication has a section on box columns of wood that may be used to some advantage for certain kinds of construction. Good design data and practical considerations are given in this volume.

Tests and reports on box column strengths have been made by Virginia Polytechnic Institute, ' Wood Box Columns and Their Design."

MECHANICALLY JOINED FRAMING

In recent years small sawn lumber pieces have been used extensively to form trussed rafters for houses, apartments, and courts. Connections may be timber connectors, or often joints are nailed. Seasoned and partially seasoned lumber is used. Typical Designs of Tim-

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[2] Verne Ketchum, May, and Hanrahan, "Are Timber Checks and Splits Serious?" Am. Inst. Timber Construction, Inc.

[3] Verne Ketchum, "Timber Maintenance Methods," Am. Inst. Timber Construction.

ber Structures and Clear Span Teco Trussed Rafters by Timber Engineering Co. contain excellent typical details of such construction. The swing to glued laminated construction has resulted in a curtailment of mechanically joined arches. The glue joint is much stronger than any mechanical joint in timber framing.

WOOD DIAPHRAGMS[4]

Oregon Forest Products Laboratory at Corvallis, Ore., in cooperation with the California State Board of Architecture, has been engaged for several years in a test program on full size model wood diaphragms. Recently a gable section of roof framing 20-ft wide and 60-ft long was tested. Report on test results is not yet available; however, the frame stood up well and compared favorably in strength to a flat section. Excellent tests on wood diaphragms were made and recommendations published in a brochure.

Wood diaphragm actions are now accepted in Uniform Building Code and in many other codes. Extensive use is being found for diaphragms because of their excellent test performances. One user alone has installed 20,000,000 sq ft of wood diaphragms. Diaphragms are especially useful in earthquake areas.

Forest Products Laboratory at Madison, Wis., has made many tests on diaphragms and on wood shear walls. Douglas Fir Plywood Assn. has carried on for several years a test program of wood diaphragms and shear walls. Twenty-five large sections varying in size from 4 by 12 ft to 20 by 40 ft have been given vertical and horizontal destructive tests.

POLE FRAME CONSTRUCTION

There is a type of construction, sometimes called pillar construction, using treated poles set in the ground for columns and beam and girt superstructure that is now being used in many localities for farm and suburban use.

In its simplest form the pole structure is one-story and has no concrete foundations and retains the dirt floor. Floorless structures fit pole frame construction well and are suited to many farm and field uses, such as hay sheds, implement sheds, some types of barns and loafing sheds, and those various buildings that are required to merely provide cover for animals, tools, or products.

HARDWARE AND IRONWORK

There has been no radical improvement in recent years in hardware and ironwork as used in timber construction. Minor changes comprise more welded construction, new shape patterns in concealed hangers to provide an all-wood appearance when desired, and a larger use of strap and pin connections. On large bridge parallel chord trusses, the vertical stress component of the diagonal web member is now often taken directly into the vertical by a welded connection rather than being passed up and back through the chord. A great saving in chord material is thus obtained.

Steel fittings and connections are usually designed and detailed in accordance with the Steel Construction Manual of the American Institute of Steel Construction. This Institute has also embarked on the writing of a series of textbooks on Structural Shop Drafting.

Aluminum fittings are not yet economical as compared to those of steel in timber construction.

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[4] See also papers of Charles Peterson and R. P. A. Johnson, pp. 16 and 21 of this publication.

AMERICAN INSTITUTE OF TIMBER CONSTRUCTION

The American Institute of Timber Construction, an organization of timber fabricators, is now in its fifth year. This Institute is widely recognized in the construction industry, and through its Standards Committee has prepared and published much excellent timber literature. The Institute may be consulted on all problems of engineered timber construction. Here is an unbiased source of knowledge and opinions on construction for all who build with timber. These following standards are available:

1. Timber Construction Standards,
2. Inspection Manual for Structural Glued Laminated Lumber,
3. Timber Trusses and Bracing, and
4. Standard Appearance Grades of Structural Glued Laminated Lumber.

FIRE SAFETY

The timber construction interests have been impressed in recent years with the handicaps of codes and insurance rates which rule out timber where otherwise it would be selected. The industry now has in preparation "The Case for Timber" that will contain pertinent data on fire losses, comparisons with other building materials and case histories.

BUILDING CODES

In the field of standard building codes, the 1955 edition of the Uniform Building Code contains a very good section on timber construction. Timber construction is also well represented in the 1955 edition of the National Building Code as issued by the National Board of Fire Underwriters.

The Southern Building Code Congress, which publishes the Southern Standard Building Code, has recently embarked upon a program to bring its code up to date with a new 1956 edition.

Portland, Ore., has adopted this year a new building code after several years of study and preparation. Its requirements are in line with modern timber design and construction practices.

TIMBER BUILDING TYPES

Codes, in general, specify several types of timber construction such as Heavy Timber Construction, Ordinary Construction, and Wood Frame Construction.

Heavy Timber classification requires certain timber member sizes plus masonry, reinforced concrete or non-combustible exterior walls. Wood Frame classification permits smaller timber member sizes than allowed for Heavy Timber, but still requires non-combustible exterior walls. Wood Frame classification permits standard engineering members and does not require non-combustible enclosure walls. The origin of these classifications lies in the early New England textile factories built with masonry walls and post and girder interior framing, and where buildings were more than one story in height.

It is stated in National Fire Protection Assn. literature that the chief value of a fire-resistant exterior wall is important only to the spread of fires to or from adjacent buildings. Separations would serve the same purpose. The heavy timber structure of today varies in several ways from the former style in that it is seldom multi-story and in that it has long spans and often has large glued laminated girders or arches. It may or may not have masonry exterior walls. Many of the industrial plants being built today, as well as schools and industrial and commercial construction, are built in the country far from congested urban centers. Separations are frequently a natural part of the planning.

We need a thorough revision of the

Heavy Timber rulings for buildings and fire safety classifications that have been with us, unchanged, since early New England days.

RECENT IMPRESSIVE TIMBER STRUCTURES

It seems that in recent months there has been planned and built a series of monumental timber structures that have lengthened spans and have used timber in new types of work. Trusses and arches have been produced to longer spans, and timber domes to normal and record-breaking spans are now being built. Timber centering and falsework, though accessories to steel and concrete framing, are sometimes attainments in their own right. Often they rise and lower, and move sideways, or revolve to perform their special important functions.

While many new forms of timber construction depend largely on glued laminated procedures, sawn lumber still plays an important part. There follows a description of some late important timber projects.

ILLUSTRATIVE LARGE TIMBER PROJECTS

1. *Timber Highway Bridge* in the Province of Manitoba, Canada, to H15 loading required two glued laminated girders, 12-in. thick, 72-in. deep and 103-ft long, which were given a pressure preservative treatment.

2. *Snohomish County Airport* near Everett, Wash., required eleven glued laminated arches 12 by 40 in. in cross-section to produce a clear span structure 200-ft wide, 210-ft long and 55-ft high, together with additional shops and administrative units.

3. *Health and Physical Education Building* at Montana State College, Bozeman, Mont., now being erected, contains a 300-ft clear span dome framed in timber. Ribs are 7 by 16 in. glued laminated timbers.

4. *St. Margaret Mary Church*, Milwaukee, Wis., contains five 100-ft span timber three-hinged rigid frames.

5. *Warehouse* for Potash Company of America, at Carlsbad, N. M., is 180-ft wide by 600-ft long and contains about twenty-four 180-ft span timber arches.

6. *Field House* at Montana State University, Missoula, Mont., is a timber framed structure containing glued laminated arches which span 200 ft.

7. *Westchester County Airport*, White Plains, N. Y., contains timber bowstring trusses of 250-ft span, set with three trusses end to end to form a structure 750 ft in length.

8. *Jai Alai Fronton* at West Palm Beach, Fla., has twelve timber arches with a clear span of 242 ft and a rise of 74 ft. Arches are three-hinged and depth of section varies from 25 in. to 46 in. in depth. This job now holds the world's record for length of span of timber arches.

9. *Island Garden Arena*, Hempstead, L. I., is a 280-ft square timber structure that contains ten 250-ft clear span timber glued laminated arches. Arch ribs are 9 in. in width and 26 to 56 in. in depth. This project by a slight margin will take the world's record from the Jai Alai project at West Palm Beach.

10. *Minneapolis Field House* at University of Minnesota, is a 200 by 400-ft structure containing sixteen 200-ft span timber trusses. Trusses and purlins are constructed of sawn lumber.

CONCLUSION

In late years the synthetic building materials, other than wood, have multiplied in number. Plastics, aluminum, fiber glass, and cement boards are advertised extensively. Among wood and wood products we have a multitude of chip boards, boards made from seaweed, cornstalks, and straw. All these things of either natural or synthetic origin, and whether stamped, poured, extruded or laminated, do enhance the building industry and bring color and variety to the field. Many of these, however, including glued structural timbers, complement rather than compete with sawn timbers and lumber in the structural field.

Economy rests with sawn timbers in the fields of construction usage where appearance is not a prime requisite, and seasoning in place is permissible. Sawn material is not normally obtainable in fully seasoned condition in thicknesses over 2-in. nominal, though some 3- and 4-in. can be dried to special order and expense. Large sawn timber sizes, and especially long lengths, are much less readily obtainable than in former times. This is due, not to timber depletion, but to changes in logging and sawing methods to meet sustained yield requirements and to the changed market for the log, which sends the top grades to the plywood mill rather than to the sawmill.

It is when qualities not obtainable in the plain sawn material are desired that glued laminating of structural members come in to complement the line. Glued laminated members are more expensive and no doubt always will be, foot for foot, than sawn material, though considerably stronger, size for size. The material is a superior product. It is preseasoned and may be had in any size, length, curve, or shape and in strength and appearance grades to fit the individual requirement. The well-informed designer will do well to look at both materials and use them as their respective merits fit his needs and desires. Both come originally from the forest and from the saw and it is a happy circumstance that so wide a selection has become available.

Sawn lumber construction is and will, for many future years, be an important building material.

DEVELOPMENTS IN SOFTWOOD PLYWOOD DESIGN AND CONSTRUCTION

By David Countryman[1]

Synopsis

Developments within the softwood plywood industry which have affected applications of plywood in construction are reviewed. These include the activities of industry, through the Douglas Fir Plywood Assn., in developing quality standards. They also include the program of testing and research on design methods and structural applications carried on by the plywood industry and by the United States Forest Products Laboratory.

Available design procedures are mentioned and the development of working stresses traced. Application of these working stresses is outlined, using approximate design methods for tension, compression, flexure, and shear.

Common structural applications and construction practice are discussed for wall, roof, and floor sheathing, exterior applications, concrete forms, and overlaid plywood. Special structural applications for which plywood is commonly used are also listed, including shear diaphragms, built-up plywood beams, gusset plates for trusses and rigid frames, and stressed-skin panels. Several experimental types of roof structures are also mentioned, including hipped-plates, hyperbolic paraboloids, and elastic arches.

It is concluded that the availability of technical information and the development of adequate quality standards within the past two decades have contributed substantially to the acceptance of plywood as a construction material.

Developments in application of plywood to construction are best appreciated by looking back to twenty-five years ago. At that time uses of softwood plywood consisted almost entirely of crating, backing, cheap furniture, and a few industrial uses such as automobile floor boards. There was no exterior type, nor were special panels generally available. Annual production was 200,000,000 sq ft, of which only a small amount went into construction, and then for such items as door panels or cupboards. There were only a handful of mills, with little or no industry organization.

Within the next decade several developments happened which greatly altered this picture. Exterior type and sheathing grade were developed. The Douglas Fir Plywood Assn. was organized in its present form, and through it industry began investigating possible construction applications and disseminating technical data. An effective quality control program was established to assure a uniform product. Building agencies such as the Federal Housing Administration and regional building codes recognized plywood as a construction material, as did various government agencies, who specified it in large quantities during the war.

With these developments, the use of plywood in construction increased rapidly, with total production running 1,200,000,000 sq ft, $\frac{3}{8}$-in. basis, in 1940 2,550,000,000 sq ft in 1950, and estimated at 5,500,000,000 sq ft in 1956.

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[1] Chief, Engineering-Research, Douglas Fir Plywood Assn., Tacoma, Wash.

The number of producing mills increased from 28 in 1940 to 121 (in the United States) at present.

Where plywood has been used structurally in this country, it has commonly been Douglas fir, except for a few specialized uses, such as aircraft, where hardwood plywood is employed. This paper will be confined to a consideration of softwood plywood made primarily of Douglas fir, although there is a small amount made of other species also.

INDUSTRY QUALITY STANDARDS

The standard to which most Douglas fir plywood is manufactured is promulgated as Commercial Standard CS45-55. The corresponding standard for other western softwood species is CS122-56. The Douglas Fir Plywood Assn. is concerned with plywood manufactured by its members under both of these standards. In addition there is a standard for western pine plywood, numbered CS157, and one for hardwood plywood, CS35.

The Commercial Standards CS45 and CS122 specify minimum requirements as to grade and species of veneer for both faces and inner plies, as well as glue bond performance for both interior and exterior types. Also covered are such requirements as size tolerances, moisture content, inspection and grade marking (**1**: **2**).[2] Douglas Fir Plywood Assn. grade marking of each panel indicates that the panel has been produced under the quality control program of the Association, and indicates conformance with the commercial standards.

The Association quality control program is carried out by a group of highly-trained quality supervisors who have access to all manufacturing processes in every member mill and who act independently of the mill. These supervisors check all phases of production, particularly panel grading and gluing. They select samples according to a carefully designed schedule and forward them to one of the Association laboratories for glue bond testing. The mills are continually apprised of test results so that any necessary adjustments can be made. It is noteworthy that Association glue bond quality requirements are substantially higher than those for the commercial standards, so that in order to qualify for the Douglas Fir Plywood Assn. grade mark, the mill must continually exceed what may be considered as ordinary good commercial quality.

DEVELOPMENT OF DESIGN DATA

The United States Forest Products Laboratory pioneered some of the first structural applications of plywood. In 1934 it developed and tested the plywood stressed-skin panel, which was soon to be adopted by the prefabricated home industry. It also performed a series of racking tests on walls sheathed with plywood which clearly demonstrated the high shear strength and bracing potentialities of such construction. In 1941 the first working stresses and their application to plywood were published. During the war years, much basic work was done at the Laboratory on aircraft applications, both through structural analysis and by tests. This information opened the way for many new plywood applications and for refinements in existing ones.

The fir plywood industry, operating primarily through its trade association, the Douglas Fir Plywood Assn., has also developed and published information intended to facilitate proper and economical uses of plywood. In its Tacoma laboratory, the Association carries on a continuous program of testing and applied research. This research takes place in categories which may be described as appraisal, application, and improvement. Appraisal includes work on plywood's mechanical properties, such as flexure, shear, impact, and strength of various fasteners, as well as its physical properties, such as dimensional stability, thermal and vapor transmission, fire and chemical resistance, preservative treatments, and finishing. Application has included such structural developments as diaphragms and shear walls, stress-skin panels, box beams, gusset plates, and various sheathing developments. Product improvement includes a study of the effect of manufacturing processes on warping and face-checking, as well as other industry problems.

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[2] The boldface numbers in parentheses refer to the list of references appended to this paper, see p. 85.

The availability of such product information has helped to increase the acceptance of plywood by the building industry. Included are acceptances by such national agencies as the Federal Housing Administration and the Veterans Administration. The major regional codes, such as the Basic, the Uniform, and the Southern Building Code, as well as many state, county, and municipal codes, also accept plywood as a structural material. Many of these codes incorporate a requirement that the plywood be grademarked by an approved agency in order to qualify for structural uses.

PRINCIPLES OF DESIGN

Within the past two decades design with plywood has progressed from something less than rule-of-thumb methods to procedures which may involve highly complex mathematical considerations. Early plywood construction applications simply involved enclosure or coverage, as might be typified by doors or cabinets, where loads and strength were not formally considered.

Design procedures which were developed during the war permitted the solution of the extremely complicated structural problems involved in analysis of plywood aircraft. These procedures make allowance for such factors as the species, construction, specific gravity, moisture content, and angle of adjacent plies in computing ultimate stresses. They may be used to solve various loading conditions, including compression, tension, and shear, or any combination thereof, on elements at any angle with the face grain, elements in bending or acting as columns; flat panels, buckling criteria and strength after buckling, in compression, tension or shear, or any combination thereof, at any angle with the face grain; effect of stiffeners, allowable shear strength of webs and torsional strength and rigidity of plywood beams; plywood panels under normal loads, with and without stiffeners; curved panels in compression, tension, shear, or combination; bending, torsion, including consideration of stiffener effects; and strength of bolted or glued joints (3).

These data, although occasionally useful in structural problems, seldom warrant the additional effort and precision required. Furthermore, the grading of commercial plywood justifies precise structural analysis only in special cases. Thus for the vast majority of cases it is sufficiently accurate to employ an approximate design method, using working stresses well below the ultimate plywood strength.

Working stresses for plywood are based on the strength properties and "basic stresses" for the species of wood from which it is made, as tabulated by the U. S. Forest Products Laboratory (4). These basic stresses are determined from the average ultimate stress obtained from tests on clear, green wood, reduced to allow for variability, long-time loading, and a true factor of safety. For shear, an additional reduction included is to account for the development of seasoning checks in lumber, a reduction that is not applicable to plywood. Only the elastic modulus is not reduced; instead its approximate average value is used in design.

There are two important adjustments of the basic stresses required to obtain satisfactory working stresses. One is for moisture content during service and the other for grade of veneer. Stresses for certain strength properties (bending, compression, tension) are increased for the effect of drying. Basic stresses in shear and elastic modulus are also increased, but the Douglas Fir Plywood Assn. does not yet recommend that these increases be applied to plywood until further tests can be made on plywood itself. Thus in the case of Douglas fir plywood, the elastic modulus of 1,600,000 psi is the basic value for green wood, whereas plywood is kiln-dried in manufacture. It is estimated from distribution curves of tests that approximately 85 per cent of dry Douglas fir has an elastic modulus equal to or higher than 1,600,000 psi.

Reductions for grade are based on estimated average effects of combinations of defects permitted within the veneer grade specified. These grade factors are applied to the basic stresses for the various plywood grades for all strength properties except elastic modulus and bearing perpendicular to the grain.

Where mixed species are allowed, as in western softwood plywood made under CS122, the stress for the weakest species permitted is listed. When these species are permitted only in some plies, as in certain finish grades of interior type made under CS45, which have Douglas fir faces but permit other western softwoods in inner plies, strength of the lower species is applied under those conditions which stress the inner plies, while the higher species value is applied to the faces. Stresses such as shear, which affect all plies equally, are prorated in accordance with the panel construction.

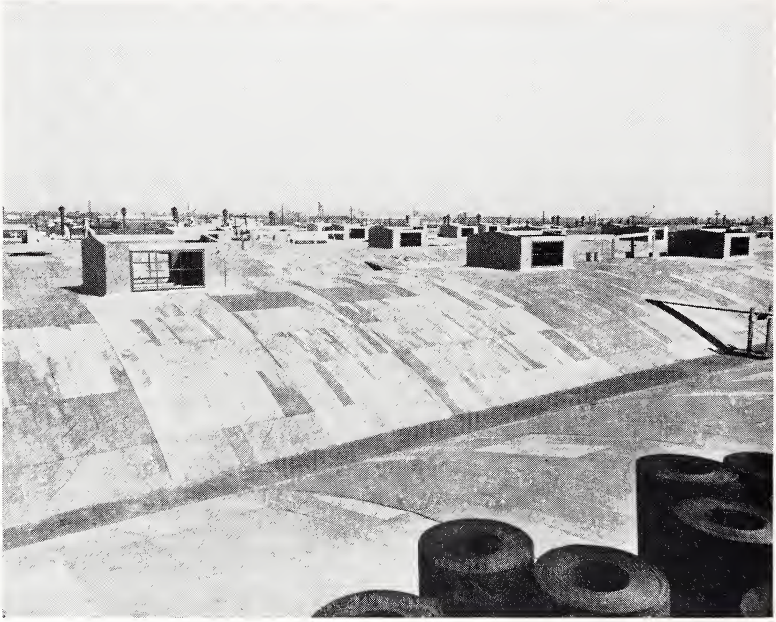

FIG. 1.—The Roof of this 90,000 sq ft Warehouse Addition was Sheathed with Plywood on Joists Between Bowstring Trusses. Roof was designed as a horizontal shear diaphragm to carry 260 lb per lin. ft.

Since plywood is not a homogeneous material, these working stresses must be applied with due regard for that portion of the cross-section that is stressed. For example, in tension, compression, and flexure, only those plies whose grain runs parallel with the stress are considered as carrying load. In actuality, those plies that are perpendicular to the stress also carry load but in proportion to their stiffness relative to the parallel plies. Since this relative stiffness amounts to about 5 per cent for tangentially-cut Douglas fir, these plies may conservatively be ignored. Thus in tension or compression, working stresses are applied to the area of the plies parallel with the load, while in bending the moment of inertia and the section modulus of the plies parallel with the span are considered. An exception is bending perpendicular to the face grain of three-ply, where stiffness of the faces becomes a significant factor. Thus it is necessary to know the construction of the plywood, that is, the thickness, grade, and species of each veneer, for proper design.

When plywood is stressed in tension or compression at a 45 deg angle with the face grain, the entire cross-section is considered, but at a lower allowable stress than when parallel.

Shear stresses in plywood are of two types, either one of which may be critical in design. One kind is that perpendicular to the plane of the panel, such as horizontal shear in a built-up beam having a plywood web. This shear is computed over the full cross-sectional area of the panel, using an allowable stress, when the direction of stress is parallel or perpendicular to the face grain, of twice the basic shear stress for wood (with suitable reduction for grade). When the shear is at 45 deg to the face grain, four times the basic shear stress is allowed.

The second kind of shear is that in the plane of the panel. In plywood, this shear tends to cause the fibers in the ply at right angles to the shearing force to roll against adjacent plies. Resistance of plywood to this rolling shear is about one-third that offered by wood parallel to the grain. However, because of beam-checking and other factors not present in plywood, which reduce the basic stress for lumber considerably, the allowable shear stress for plywood in rolling shear is taken as three-fourths of the basic shear stress for wood (with suitable grade reductions).

Normally, plywood adhesives bond the veneers together so as to develop its full strength, so that the wood fibers, rather than the adhesive, limit allowable shear values.

Occasionally the allowable bearing stress perpendicular to the plane of the panel may be required, as in determining bearing strength of fastenings. This value is determined from the basic stress for wood in compression perpendicular to the grain, increased for drying, but not adjusted for grade.

Detailed values of section properties, working stresses, and their methods of application to plywood are not given in this paper, as they are available elsewhere (**5**).

COMMON STRUCTURAL APPLICATIONS

Sheathing:

Probably the most common structural application of plywood is for sheathing—roofs, walls, and floors. The panel used most frequently for this purpose is an unsanded interior type, C-D grade,[3] which, when manufactured under Douglas Fir Plywood Assn. quality supervision, is known as PlyScord. Other grades may also be used as sheathing, including exterior type, western softwoods species, or even finish-grade panels, where a combination structural and appearance value is desired, as for wall panelling or siding.

For side wall sheathing, panels may be applied either vertically or horizontally. Where studs are spaced 16 in., a $\frac{5}{16}$-in. panel is recommended by DFPA, and with studs at 24 in., $\frac{3}{8}$-in. plywood is permissible. Siding and shingles are applied conventionally and may be nailed directly to the plywood. Building paper is not required unless a water permeable siding, such as masonry, is used. When sheathing is properly nailed, no other bracing is normally required, as the diaphragm action of the panels develops ample strength and rigidity (6).

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[3] See CS45–55 (**1**) for description of grades A, B, C, and D grades of veneer.

Allowable spans for roof sheathing are limited by an allowable deflection of $\frac{1}{240}$ of the span, assuming the plywood face grain perpendicular to the supports, and the panel continuous over two or more spans. However, for many combinations of roof loads and plywood thicknesses, the maximum recommended span is limited by the effect of concentrated loading, such as may be encountered during construction or in maintenance. For example $\frac{3}{8}$-in. fir plywood supported on rafters spaced 24-in. centers requires a load of 46 lb per sq ft to cause a deflection of $\frac{1}{240}$ of the span, yet the 24-in. spacing is the maximum recommended, even for roof loads of 20 lb per sq ft.

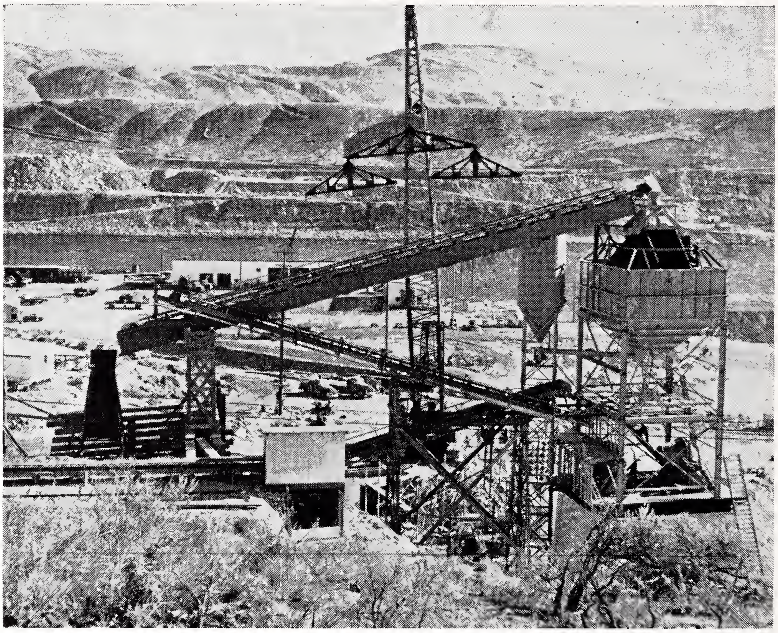

FIG. 2.—Gravel Conveyor Belt for Concrete Mixing Plant at Chief Joseph Dam on the Columbia River Is Supported on two 120 ft Plywood I-beams Spanning 90 ft Beams are 4 ft Deep, Spaced 3 ft 3 in. Apart, With 2 by 6 Glued and Bolted Lumber Flanges and 1 in. Plywood Webs and Are Designed for a Load of 250 plf. Deflection Under 9000 lb Center Test Load was $1\frac{5}{8}$ in.

Blocking or other suitable panel edge support is also required when spans exceed a certain value for each plywood thickness. This requirement is also limited by strength and stiffness under concentrated loads applied at the panel edges. It is designed to provide a roof that will be safe under traffic and which will protect the finish roofing from damage. It may be affected by the character of the finish roof. For example a steeply pitched roof covered with asphalt shingles is less likely to be damaged by traffic than a flat roof with light built-up roofing.

Plywood floors are limited not by bending strength but by stiffness under concentrated loads, such as traffic, which is largely a matter of customer acceptance. Minimum thickness may also be limited by nail holding power for finish hardwood flooring. With joists spaced 16 in. on center, $\frac{1}{2}$-in. Douglas fir or $\frac{5}{8}$-in. western softwood plywood may be used, provided its grain is perpendicular to supports. With joists at 24 in., $\frac{3}{4}$ in. in these species is acceptable. These combinations are all adequate for uniform loads of at least 100 lb per sq ft within allowable fiber stresses and a deflection limitation of $\frac{1}{360}$ of the span, even where the finish flooring has no structural value. However, where resilient flooring is applied directly to the plywood, solid blocking is required under panel edges, so as to prevent differential deflection from damaging the flooring.

A recent development in plywood floors is a thick combination subflooring and underlayment panel designed to cover girders set 48 in. on center. This product, known as 2.4.1 plywood,[4] is $1\frac{1}{8}$ in. thick, having Douglas fir faces and permitting inner plies of western softwoods species. It is applied in 4 by 8-ft panels across two 48-in. spans, with 2 by 4-in. blocking beneath the edges. Tests indicate a load capacity of 100 lb per sq ft at a deflection of $\frac{1}{360}$ of the span and without exceeding the allowable fiber stress. This panel has been found to make an adequate and economical base for direct application of any type of residential finish floor.

Exterior Applications:

Exterior type plywood today is commonplace, yet it was only a little more than 20 yr ago that it was first produced. In the construction field, where it is frequently employed as an exterior wall covering, it sometimes also acts as sheathing, so that when adequately nailed, other wall bracing may be omitted. It may be applied as panelling or lap-siding, either to sheathed or unsheathed walls.

There are important structural uses for the exterior type in agricultural buildings, where it has been used, for example, to form the walls of round bins and silos carrying both the imposed horizontal and vertical loads. Its light weight and high rigidity make it valuable in marine uses, for boat hulls, decks, and bulkheads of sizable commercial and military as well as pleasure craft. Quantities of exterior plywood have been used for both inside and outside lining of railroad box and refrigerator cars.

A number of plywood applications have been made possible because of the fact that exterior type can be pressure treated with oil or water-borne preservatives to resist fire and decay. There is evidence that plywood is readily and completely penetrated by such preservatives through the end grain wood exposed on the panel edges.

Concrete Forms:

Concrete forms are a construction application for which plywood is well-suited, and for which it has been accepted for many years. The special grade developed for this use is known as Plyform, when manufactured under Douglas Fir Plywood Assn. quality supervision, and is a sanded B-B grade, of either interior or exterior type, the latter being more commonly used. Most common thicknesses are $\frac{5}{8}$ in. and $\frac{3}{4}$ in., depending on design of forms, including stud spacing and concrete pressure.

It is customary to vary these factors so as not to exceed a deflection in the plywood of $\frac{1}{270}$ of the stud spacing, making due allowance for the effect of panel continuity and decrease in stiffness with wetting of the wood.

To facilitate stripping, plywood forms are usually mill-oiled lightly, using a No. 100 pale oil. Some contractors prefer to use one of the form lacquers available instead, in which case the plywood is specified as un-oiled.

Most plywood forms are built in-place, but there are a number of panellized systems, usually available on a rental basis, which facilitate setting and removal. An interesting recent development is a system employing $1\frac{1}{8}$ in. thick plywood in which metal ties which also act as spreaders combine with special hardware to clamp the panels together so that no framing studs or walers are required.

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[4] Trade name copyrighted by Douglas Fir Plywood Assn., Tacoma, Wash.

Overlaid Plywood:

A modified plywood product that has been increasingly used in construction and elsewhere is overlaid plywood. This is an exterior plywood covered on one or both faces with a phenolic resin-impregnated carrier sheet. The resin is set under heat and pressure, at the same time that the overlay is bonded to the wood. Two types are produced—one is called high-density; the other, medium density—depending on the resin content of the carrier sheet.

The purpose of the overlay is to improve the surface characteristics of the panel, particularly smoothness, hardness, and paintability. They also produce a minor increase in structural properties across the grain of the panel, and generally increase its resistance to water absorption and vapor transmission.

Principal constructional uses for overlaid plywood have been as exterior panelling or siding, because of improved surface weathering, and as concrete forms, particularly for architectural concrete, because of improved smoothness.

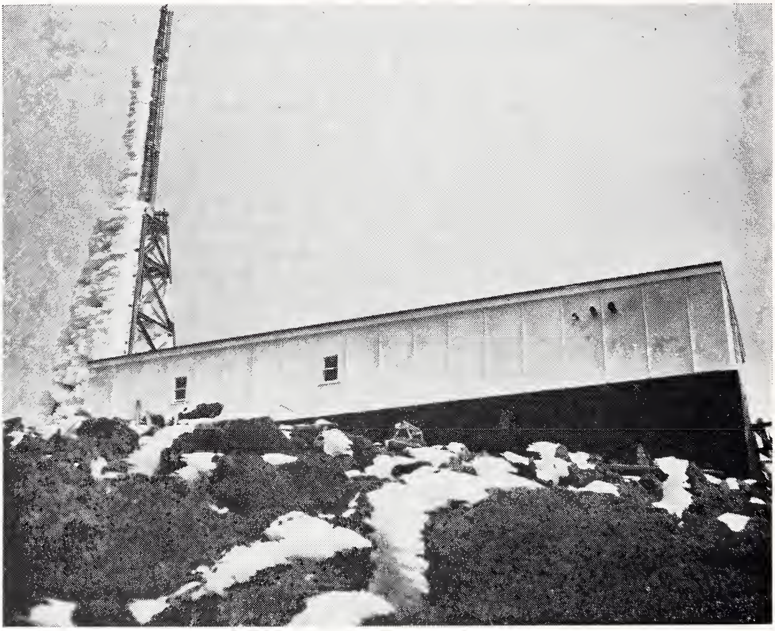

FIG. 3.—Television Transmission Studio on Summit of Mt. Washington in New Hampshire was Designed to Withstand Winds up to 230 mph and Temperatures Many Degrees below Zero. It was prefabricated using stressed-skin plywood panels.

SPECIAL STRUCTURAL APPLICATIONS

The properties of plywood make it particularly suitable for certain special structural applications. Some of the important properties include the large panel sizes available, with any lengths practical to ship, light weight, and multi-directional strength, particularly good shear strength and nail-bearing properties.

Diaphragms:

One example of this application is the plywood shear diaphragm, which consists of framing members covered with plywood. It may be horizontal, such as a roof or floor, or vertical, such as a wall or partition. Its function is to brace a building against lateral wind or seismic forces, and to transmit these forces into the foundations. In this respect it replaces the ordinary bracing system of the building, which usually consists of diagonal members. Since the building must be sheathed in any event, use of the sheathing to perform the bracing function makes for economy plus a more rigid building.

In diaphragms the plywood sheathing acts as a fully shear-resistant web, as in a girder. Flexural stresses are carried by continuous boundary framing members. Intermediate framing members, which may be of wood or steel, stiffen the diaphragm against buckling and serve as splicing members at plywood panel edges. Panel edges between framing members may be left unblocked, but the diaphragm is stronger and stiffer if they are spliced, as when nailed to solid blocking, or with cleats or similar shear-resistant connections.

Strength of horizontal plywood diaphragms has been determined from tests on some 25 full-size members made by the Douglas Fir Plywood Assn., as well as by several specimens made and tested by the Oregon Forest Products Laboratory. Testing has also been carried out on vertical plywood diaphragms by both agencies.

Plywood diaphragms range in size from a single 4-ft panel used as a shear wall up to roofs as large as 500,000 sq ft acting as a single diaphragm. Magnitude of shears induced by horizontal forces depends on the shape and construction of the building, as well as the assumed loading. Roof shears are frequently less than 300 lb per linear ft of width, while wall shears may be higher, particularly where the wall is braced by isolated piers of narrow width. Plywood diaphragms may be designed for shears ranging up to 820 lb per linear ft under accepted tabulated code values (**9**), or even up to the shear strength of the plywood itself, providing it is adequately fastened to the framing members, prevented from buckling, and the diaphragm sufficiently anchored.

Just as with beams, diaphragm design may be limited by allowable deflection of attached elements, such as masonry sidewalls. An accurate method exists for calculating the elastic deflection of plywood diaphragms (**10**).

Beams:

Plywood beams are built-up members consisting of one or more vertical plywood webs to which lumber flanges are attached along top and bottom edges. They may be fabricated with nails, bolts, or glue, or with combination of these. They combine stiffness with light weight and are adaptable to both shop and site fabrication. They are convenient for covering long spans where only small pieces of solid lumber are available, or where shipping weight is important, or where minimum shrinkage or good apperance is desirable. They have been used in spans ranging up to 100 ft for such purposes as movable catwalks or conveyor belt supports on heavy construction jobs, while in buildings they have served as lintels, ridge beams, and floor and roof girders

Their design differs from that of a solid wood beam in that a form factor is applied to the allowable bending stress to express the reduction in allowable compression flange stress caused by reduced support of the individual wood fibers. Also the attachment of the plywood web to the flange must be designed for shear, and the web itself must carry the horizontal shear. Stiffeners are spaced so as to prevent web buckling, and splices provided in flanges and webs (5).

Gusset Plates:

Because it has good strength properties in all directions, plywood is used for gusset plates to facilitate joint design with trusses and rigid frames. The gusset plates permit framing members to be coplanar, which eliminates joint eccentricity caused by lapped members and simplifies layout of attached pieces.

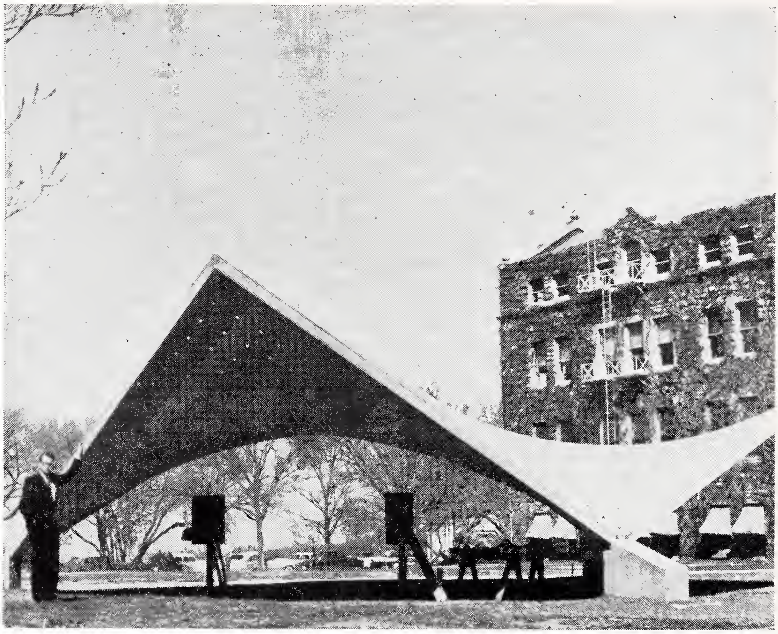

FIG. 4.—Experimental 20 by 40 ft Hyperbolic Paraboloid Roof Structure Built at Kansas University Consists of two Layers of ¼ in. Plywood Nail-Glued to Plywood Box Beam Perimeter Members.

For light clear-span residential roof trusses, ⅜-in. or ½-in. plywood may be placed on both sides of 2-in. wood framing members, using nails or nail-gluing. Considerable efficiency in nailing can be obtained by using nails long enough to penetrate the assembly, clinching them on the gusset plate so that they function in double shear.

Also used extensively are similar light roof trusses which employ open plywood webs glued to the chords on one or more sides. Such webs do not form a continuous shear-resistant member such as those in a plywood beam but are larger than simple gusset plates, and transfer bending moment. Such trusses are statically indeterminate, and their design has so far been based on experimental stress analysis with strain gages.

For long-span wood trusses and rigid frames, plywood gussets have been used in thicknesses ranging up to 3⅝ in., generally with bolts or timber connectors. One fabricator has developed an ingenious rigid frame consisting of solid lumber columns and beams rigidly joined by a single plywood gusset plate of thickness equal to that of the lumber and lying in its plane. The gusset and lumber pieces are scarf-jointed and glued together, employing a reverse scarf joint.

Stressed-Skin Panels:

A stressed-skin panel is an element in which the covering acts integrally with the framing members to resist stresses. They are used extensively in prefabricated building construction for walls, floors, and roofs because they permit thinner and shallower framing members than conventional frame construction, thus reducing handling weight and volume. Plywood must be glued to both sides of the framing members, as mechanical fasteners alone are not sufficiently rigid in shear to develop reliable stressed-skin action.

Recognized design methods exist by which strength and stiffness under flexural loading may be calculated (5). These take into account possible buckling of plywood between longitudinal members as well as the shear between plywood and framing. Panels have also been tested in racking and compression.

Most stressed-skin panels have been used for dwellings, so that spans normally range up to 12 ft. A recent development has been the use of such panels for other types of structures, such as schools and commercial buildings, with a consequent increase in span to 16 or 20 ft. The only limit to possible length seems to be the practical one of handling, with no apparent reason why spans of 32 ft or more should not be used, provided they prove economical.

Potential Structural Applications

Besides the above familiar applications there are several of fairly recent development which may indicate important future uses.

Thin Shells:

One of these types is the thin shell, particularly the hyperbolic paraboloid, which requires no compound curvature in the panels. Under uniform loads such a form develops only membrane stresses, with little or no flexural action. Thus even thin plywood can be used on surprisingly long spans, provided panels are spliced to transmit tension, compression, and shear. Such splicing can be accomplished in one direction by using panels scarf-jointed to full length, and in the other by mechanical fasteners, using lapped joints, or an extruded metal connector. An alternate method is to use two layers of thinner plywood running at right angles. Limited experience with these forms indicates that they make very efficient use of material and will be entirely practical as long-span roof structures.

Hipped Plates:

Another roof structural system where plywood is particularly applicable because of its excellent diaphragm properties is the so-called "hipped plate." In a simple form, this may consist of two inclined plates, forming a pitched roof, stiffened with longitudinal girders at the two eaves and joined by two end diaphragms to form an inverted open box or tent-shaped structure. The entire assembly may be set on a post at each of the four corners. Spans of considerable length can be attained because of the rigidity of the pitched plates which act as diaphragms to resist deflection in their own plane. Each plate offers support to the edge of adjoining inclined plates; shears are transferred to the end walls by diaphragm action.

In known applications with wood framing and plywood sheathing, bearing walls have been substituted along two sides for the longitudinal girders, but the diaphragm action of the roof slabs has still been utilized to eliminate cross-ties (11).

Elastic Arches:

A third type of application is the use of plywood as an elastic arch (12). This is made practical through the availability of long lengths of plywood scarf-jointed at the mill so as to develop their full strength. In such construction a panel of plywood is sprung to a curve and held thus with ties or abutments. Such a shape is capable of carrying heavy uniform loads, but is susceptible to unbalanced or concentrated loads unless stiffened. Stiffening members may commonly be light lumber pieces, 1 or 2 in. thick, placed longitudinally, which also serve as splicing members for adjacent panels, or the panel may be stiffened by using a thick but lightweight honeycomb core between two plywood layers, to form a sandwich panel.

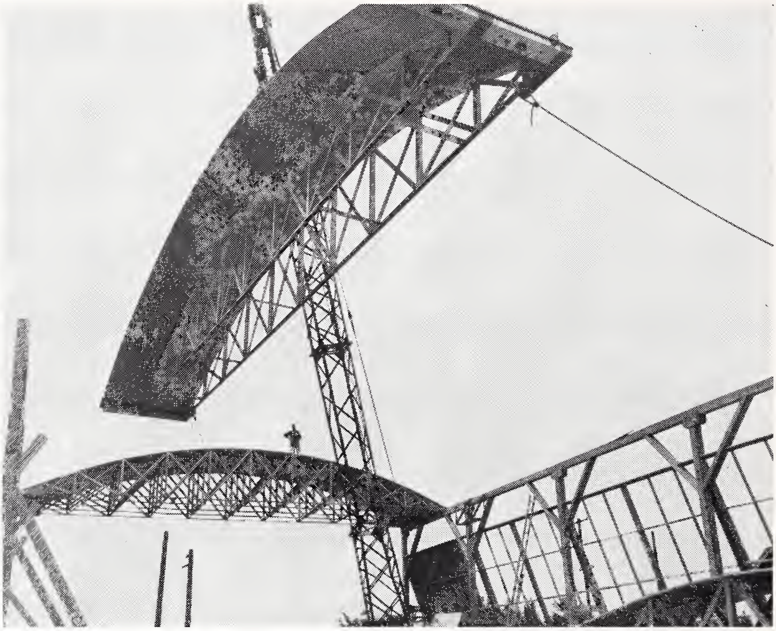

FIG. 5.—Sixty-Foot Span Plywood Roof Trusses are Erected in Pairs to Form a 240 by 360 ft Warehouse. Plywood in 4 by 62 ft Lengths Acts as Combination Sheathing and Upper Chord of Light Wood and Steel Trusses Spaced 4 ft on Centers.

An interesting application of an elastic arch is a 60-ft span curved roof truss in which $\frac{3}{4}$ in. plywood panels 62 ft long form both the upper chord and the roof sheathing (13). The panels are supported by light wood stiffening trusses spaced at 4 ft, which also support the steel bar tension chord. Strength of the panels is developed by special full-width end bearing assemblies. This truss has been tested to 96 lb per sq ft and deflected only $\frac{1}{1150}$ of the span under the design load of 35 lb per sq ft.

CONCLUSION

Within two decades softwood plywood has become recognized as a major material of construction. Its versatility is demonstrated by the variety of uses to which it has been put. Not only are new uses being continually discovered, but an improved understanding of the product gained by constant research is bringing about greater efficiency in familiar applications.

As more emphasis is put on structural research and development of new products, plywood will continue to serve the construction industry by offering ever greater economy coupled with increasing freedom of design.

REFERENCES

(1) U. S. Dept. of Commerce, "Commercial Standard CS45-55 for Douglas Fir Plywood," Ninth Ed. U. S. Govt. Printing Office (1955).

(2) U. S. Dept. of Commerce, "Western Softwood Plywood, Commercial Standard CS122-56," U. S. Govt. Printing Office (1956).

(3) Munitions Board, Aircraft Committee, "Design of Wood Aircraft Structures," ANC-18, U. S. Govt. Printing Office, June, 1951.

(4) U. S. Forest Products Laboratory, "Wood Handbook," *Handbook No. 72*, U. S. Govt. Printing Office (1956).

(5) "Technical Data on Plywood" Douglas Fir Plywood Assn., Tacoma, Wash., (1948).

(6) Housing and Home Finance Agency, "Expected Resistance to Racking of Douglas Fir Plywood-Sheathed Walls," *Technical Bull. No. 13*, U. S. Govt. Printing Office, March, 1950.

(7) "Information on Plywood Sheathing," Douglas Fir Plywood Assn., Tacoma, Wash., March, 1955.

(8) David Countryman, "The 2.4.1 Plywood Floor System," *Laboratory Report No. 72*, Douglas Fir Plywood Assn., Tacoma, Wash. (1956).

(9) Uniform Building Code, Pacific Coast Building Officials Conference, Ed., Vol. I, Sec. 2511 (1955).

(10) David Countryman, "Lateral Tests on Plywood Sheathed Diaphragms," *Laboratory Report No. 55*, Douglas Fir Plywood Assn., Tacoma, Wash. (1952).

(11) "New Economy Roof Construction Uses Plywood," *The Timberman, 23rd Annual Plywood Review*, Vol. LV, No. 3, Jan., 1954.

(12) G. M. Rapp, "Lightweight Prestressed Wood Arches as a New Type of Roof Construction," John B. Pierce Foundation, Raritan, N. J., *Journal*, Forest Products Research Soc. Vol. II, No. 5, Dec., 1952, pp. 164–172.

(13) "Cost Economies, New Markets, Promised by New Plywood Truss," *The Timberman, 25th Annual Plywood Review*, Vol. LVII, No. 3, Jan., 1956.

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